EARTH DAMS

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A STUDY

BY

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EARTH DAMS

CHAPTER I.

Introductory.

The earth dam is probably the oldest type of dam in existence, antedating the Christian Era many hundreds of years. The literature upon this subject is voluminous, but much of it is inaccessible and far from satisfactory. No attempt will here be made to collate this literature or to give a history of the construction of earth dams, however interesting such an account might be. The object will rather be to present such a study as will make clear the application of the principles underlying the proper design and erection of this class of structures. In no way, therefore, will it assume the character or dignity of a technical treatise.

Dams forming storage reservoirs, which are intended to impound large volumes of water, must necessarily be built of considerable height, except in a very few instances where favorable sites may exist. Recent discussions would indicate that a new interest has been awakened in the construction of high earth dams. As related to the general subject of storage, it is with the high structure rather than the low that this study has to do. To the extent that "the greater includes the less," the principles here presented are applicable to works of minor importance.

Many persons who should know better place little importance upon the skill required for the construction of earthwork embankments, considering the work to involve no scientific problems. It is far too common belief that any ordinary laborer, who may be able to use skillfully a scraper on a country road, is fitted to superintend the construction of an earth dam. It has been said that the art of constructing earth dams is purely empirical, that exact science furnishes no approved method of determining their internal stresses, and that in regard to their design experience is much more valuable than theory. When the question of stability is fully taken into consideration, it certainly requires a large amount of skill successfully to carry out works of this character.

Extreme care in the selection of the site, sound judgment in the choice of materials and assiduity in superintending the work while in progress, are all vitally essential.

Classification of Dams.

Dams may be classified according to their purpose as diverting dams or weirs and as storage dams. The former may be located upon any portion of a stream where the conditions are favorable, and the water used for manifold purposes, being conveyed by means of canals, flumes, tunnels and pipe lines to places of intended use. These dams are generally low and may be either of a temporary or permanent character, depending upon the uses to which the water is put. Temporary dams are made of brush, logs, sand bags, gravel and loose rock. The more permanent structures are built of stone and concrete masonry.

Storage dams may be classified according to the kind of material entering into their structure, as follows: (1) Earth; (2) Earth and Timber; (3) Earth and Rock-fill; (4) Rock-fill; (5) Masonry; (6) Composite Structures.

Low dams forming service reservoirs for domestic water supplies and for irrigation comprise by far the most numerous class. They are not designed to impound a large volume of water and therefore may be built across a small ravine or depression, or even upon the summit of a hill, by excavating the reservoir-basin and using the material excavated to form the embankment. These reservoirs may be used in connection with surface or gravity systems, artesian wells, or underground supplies obtained by pumping. The dams forming these reservoirs being of moderate size and height may vary greatly in shape and dimensions. The form may be made to suit the configuration of the dam site. When the earthwork requires it, they may be lined with various materials to secure water-tightness. Often such dams are made composite in character, partly of earth and partly of masonry or some other material. They are also frequently accompanied by numerous accessories, such as settling-basins, aerating devices and covers, which present a diversity in form and appearance. A presentation of the different types of dams thus employed, with a discussion of the questions pertaining to utility in design and economy in construction, would be exceedingly valuable and of general interest. Service-reservoirs will receive only a passing notice, with the hope expressed that some competent authority will discuss them in the future.

CHAPTER II.

Preliminary Studies and Investigations.

The preliminary studies and investigations which should be made prior to the construction of any dam for the storage of water have to do with (1) the Catchment Area, (2) the Reservoir basin, and (3) the Dam site.

Catchment Area.

It is thought desirable to define a number of terms as we proceed, for the purpose of correcting erroneous usage and for a clearer understanding of the subject. The catchment area of a reservoir is that portion of the country naturally draining into it. The watershed is the boundary of the catchment area and may be correctly defined as the divide between adjacent drainage systems. In regard to the catchment area it is necessary to determine:

1. Its extent and area in square miles.
2. Its topography or the character of its surface.
3. Its hydrography or precipitation and run-off.
4. Its geology, or the character of its soils and subsoils, and the nature and dip of its rock strata.
5. Its flora, or the extent to which it is clothed with forest trees or other vegetation.

All of these elements affect the volumes of maximum run-off, which is the one important factor in the construction of earth dams that must not be underestimated.

If the proposed dam or reservoir is to be located upon a main drainage line; that is, upon a river or stream, it is necessary to know both the flood and low-water discharge of the stream. Frequently no reliable data on this subject are available, and the engineer must then make such a study of the whole situation as will enable him to estimate the minimum and maximum flow with considerable accuracy.

There are numerous factors entering into the solution of this first problem, such as the shape and length of the catchment area, its general elevation, the character of its surface, whether mountainous, hilly or flat, barren or timbered.

Good topographic maps, if available, furnish valuable data on

these subjects and it is to be regretted that only a comparatively small portion of the United States has been thus mapped in detail.

The results of stream measurements, if any have been made in the catchment basin, are especially important: These are usually few in the high areas, on account of their inaccessibility. The year 1902 marked a notable beginning of such measurements in California. In many parts of the arid region of the United States, the best storage-sites are situated in the upper or higher portions of the drainage systems. This is especially true of the streams on the Pacific Slope having their source in the High Sierras. As regulators of stream-flow and for power purposes such storage is peculiarly valuable, while storage for irrigation and domestic uses may be located nearer the valleys and the centers of population.

Frequently the engineer is required to build his dam where no such data are available. In such instances he should endeavor to secure the establishment of rain gages and make measurements of the flow of the main stream and its principal tributaries at various places to obtain the desired information. Even this may not suffice, owing to the limited time at his disposal, and he must resort to the use of certain empirical rules or formulas, and make such comparisons and deductions from known conditions and results as will best answer his purpose.

The engineer should know, approximately at least, the normal yield of the catchment area, the duration of the minimum and maximum seasonal flow, and the floods he may have to provide against during the construction of his dam. These data are absolutely necessary to enable him to provide ample wasteways for his reservoirs. Many of the failures of earth dams have been the result of overtopping the embankment, which would have been averted by an ample wasteway. The most notable example of this kind in recent years was that of the South Fork Dam, at Conemaugh, Pennsylvania, in 1889, resulting in what is generally known as the "Johnstown Disaster."

There are several empirical rules and formulas for calculating the run off from catchment areas and for determining the size of spillways necessary to discharge this flow with safety to the dam. The proper formula to apply in any given case, with the varying coefficients of each, involves a thorough knowledge on the part of the designing engineer of the principles upon which the different factors are based.

It is unwise and often hazardous to make use of any important

hydraulic formula without knowing the history of its derivation. Experiments are not always properly conducted, and often the deductions therefrom are unreliable. A presentation and discussion of these formulas would require more space than can be given in this study, and the technical reader must therefore consult for himself, as occasion may require, the various authorities cited. Formulas for the discharge or run-off from catchment areas, as determined by Messrs. Craig, Dickens, Ryves and others, are given by most writers on the subject of hydraulics.

Reservoir Basin.

The next subject of inquiry relates to the reservoir basin. It is necessary that its area and capacity at different depths should be definitely known, and this information can only be obtained by having the basin surveyed and contoured. A map should be made showing contours at intervals of 2 to 10 ft., depending upon the size of the basin and the use to which the reservoir is to be put. Reservoir basins have been classified according to their location as follows:

1. Natural lakes.
2. Natural depressions on main drainage lines.
3. Natural depressions on lateral drainage lines.
4. Arbitrary and artificially constructed basins.

Natural lakes may need to be investigated more or less thoroughly to determine the character of their waters, whether saline, alkaline or fresh. It may also be necessary to know their normal depth and capacity, and to make a study of their outlet if they have one. In some instances the storage capacity of a lake may be enormously increased by means of a comparatively low and inexpensive embankment.

The area of reservoir basin, mean depth, temperature of the water, exposure of wind and sunshine, losses by seepage and evaporation, all have a bearing upon the available water supply and influence the design of the dam and accessories to the reservoir.

In determining the character and suitability of materials for constructing a dam it is necesary to make a careful study of the soil and geological formation. This is best accomplished by digging numerous test pits over the basin, especially in the vicinity of the proposed dam site; borings alone should never be relied upon for this information. For such an investigation the advisability of borrowing material for dam construction from the reservoir basin is

determined. The porous character of the subsoil strata, or the dip and nature of the bed rock, may forbid the removal of material from the floor of the basin, even at a remote distance from the dam site.

The area to be flooded should be cleared and grubbed more or less thoroughly, depending again upon the use for which the water is impounded. In no instance should timber be left standing below the high water level of the reservoir; and all rubbish liable to float and obstruct the outlet tunnel and spillway during a time of flood should be removed.

The accessories to a reservoir, to which reference has been made, may be enumerated as follows:

1. Outlet pipes or tunnel.
2. Gate tower, screens and controlling devices.
3. Sluiceways for silt or sand.
4. Wasteway channel or weir.
5. Cover, settling basin, aerating devices, etc.

Some of these are necessary and common to all classes of reservoirs, while others are employed only in special cases, as for domestic water supplies. All reservoirs formed by earth embankments must have at least two of these, namely a wasteway, which is its safety valve, and outlet pipes or outlet tunnel.

It may be stated that the proper location and construction of the outlet for a reservoir are of vital importance, since either to improper location or faulty construction may be traced most of the failures of the past. It is almost impossible to prevent water under high pressure from following along pipes and culverts when placed in an earth dam. The pipes and culverts frequently leak, and failure ensues. Failure may result from one or more of the following causes:

1. By improper design and placement of the pudddle around the pipes.

2. By resting the pipes upon piers of masonry without continuous longitudinal support.

3. By reason of subsidence in the cuts of the embankments and at the core walls, due to the great weight at these points.

4. Leakage due to inherent defects, frost, deterioration, etc.

Mr. Beardsmore, the eminent English engineer who built the Dale Dyke embankment at Sheffield which failed in 1864, and who was afterwards requested to study and report upon the great reservoirs in Yorkshire and Lancashire, said, after examination and care-

ful study of reservoir embankment construction, that "in his opinion there were no conditions requiring that a culvert or pipes should be carried through any portion of the made bank." The writer would go even further and say that the only admissable outlet for a storage reservoir formed by a high earth dam is some form of tunnel through the natural formation at a safe distance from the embankment.

Dam Site.

The third preliminary study (that relating to the dam site itself) will be considered under three heads:

1. Location.
2. Physical features, materials, etc.
3. Foundation.

LOCATION.—The location for a dam is generally determined by the use which is to be made of it, or by the natural advantages for storage which it may possess. If it be for water power it is very frequently located upon the main stream at the point of greatest declivity. If for storage it may be, as we have seen, at the head of a river system, on one of its tributaries, or in a valley lower down.

The type of dam which should be built at any particular locality involves a thorough knowledge, not alone of the catchment-area and reservoir basin, but also accurate information regarding the geology of the dam site itself. It would be very unwise to decide definitely upon any particular type of dam without first obtaining such information. Too frequently has this been done, causing great trouble and expense, if not resulting in a total failure of the dam.

The conditions favorable for an earth dam are usually unfavorable for a masonry structure, and vice versa. Again, there may be local conditions requiring some entirely different type.

Dams situated upon the main drainage lines of large catchment areas are usually built of stone or concrete masonry, and designed with large sluiceways and spillways for the discharge both of silt and flood waters. It need scarcely be remarked that, as a rule, such sites are wholly unsuited to earthwork construction. It is said, however, that "every rule has at least one exception," and this may be true of those relating to dam sites, as will appear later under the head of new types.

In a general way, the location of high earth dams is governed

by the configuration of the ground forming the storage basin. It may not be possible, however, to decide upon the best available site without careful preliminary surveys and examinations of the geological formation.

All earth dams must be provided with a wasteway, ample to discharge the maximum flood tributary to the reservoir. Whatever type of wasteway be adopted, no reliance should ever be put upon the outlet pipes for this purpose. The outlet should only figure as a factor of safety for the wasteway, insuring, as it were, the accuracy of the estimated flood discharge. The safety of the dam demands that ample provision be made for a volume of water in excess of normal flood discharge. This most necessary adjunct of earth dams may be an open channel, cut through the rim of the reservoir basin, discharging into a side ravine which enters the main drainage way some distance below the dam. It may be necessary and possible to pierce the rim by means of a tunnel where its length would not prohibit such a design. Lastly, there may be no other alternative than the construction of an overfall spillway, at one or both ends of the embankment. This last method is the least desirable of any and should be resorted to only when the others are impracticable; even then, the volume of water, local topography, geology, and constructive materials at hand must be favorable to such a design. If they are not favorable it may be asked, "what then?" Simply do not attempt to build an earth dam at this site.

PHYSICAL FEATURES, MATERIALS, ETC.—An investigation of the location and the physical features of the dam site should include a careful and scientific examination of the materials in the vicinity, to determine their suitability for use in construction. An earth embankment cannot be built without earth, and an earth dam cannot be built with safety without the right kind of earth material.

Test pits judiciously distributed and situated at different elevations will indicate whether there is a sufficient amount of suitable material within a reasonable distance of the dam. The type of earth dam best suited for any particular locality, and its estimated cost, are thus seen to depend upon the data and information obtained by these preliminary studies. Economical construction requires the use of improved machinery and modern methods of handling materials, but far more important even than these are the details of construction.

FOUNDATION.—We may now assume that our preliminary studies relating to the location and physical features of the dam site are satisfactory. We must next investigate the foundation upon which the dam is to be built. This investigation is sometimes wholly neglected or else done in such a way as to be practically useless. To merely drive down iron rods feeling for so-called bed rock, or to make only a few bore-holes with an earth auger should in no instance be considered sufficient. Borings may be found necessary at considerable depths below the surface and in certain classes of material, but dug pits or shafts should always be resorted to for moderate depths and whenever practicable. Only by such means may the true character of the strata underlying the surface, and the nature and condition of the bed rock, if it be reached, become known. If a satisfactory stratum of impermeable material be found it is necesary also to learn both its thickness and extent. It may prove to be only a "pocket" of limited volume, or if found to extend entirely across the depression lengthwise of the dam site it may "pinch out" on lines transversely above or below. Shafts and borings made in the reservoir basin and below the dam site will determine its extent in this direction, knowledge of which is very important.

Fig. 1, showing a longitudinal section of the site of the Yarrow Dam of the Liverpool Water-Works, England, illustrates the necessity of such investigation. A bore hole at station 2 + 00 met a large boulder which at first was erroneously thought to be bed rock. The hole at station 3 + 50 met a stratum of clay which proved to be only a pocket.

The relative elevation of the different strata and of the bed rock formation, referred to one common datum, should always be determined. These elevations will indicate both the dip and strike of the rock formation and are necessary for estimating the quantities of material to be excavated and removed, including estimates of cost. They furnish information of value in determining the rate of percolation or filtration through the different classes of material and the amount of probable seepage, as will appear later. The cost of excavating, draining and preparing the floor or foundation for a dam is often very great, amounting to 20 or 30% of the total cost.

Fig. 2 is a transverse section of the Yarrow Dam. This particular dam has been selected as fairly representative of English practice and of typical design. It is one of the most widely known earth dams in existence.

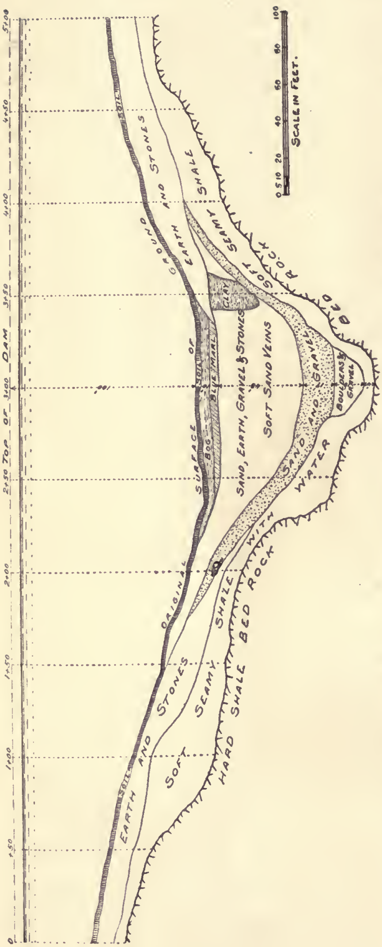


FIG. 1.—LONGITUDINAL SECTION OF YARROW DAM SITE.

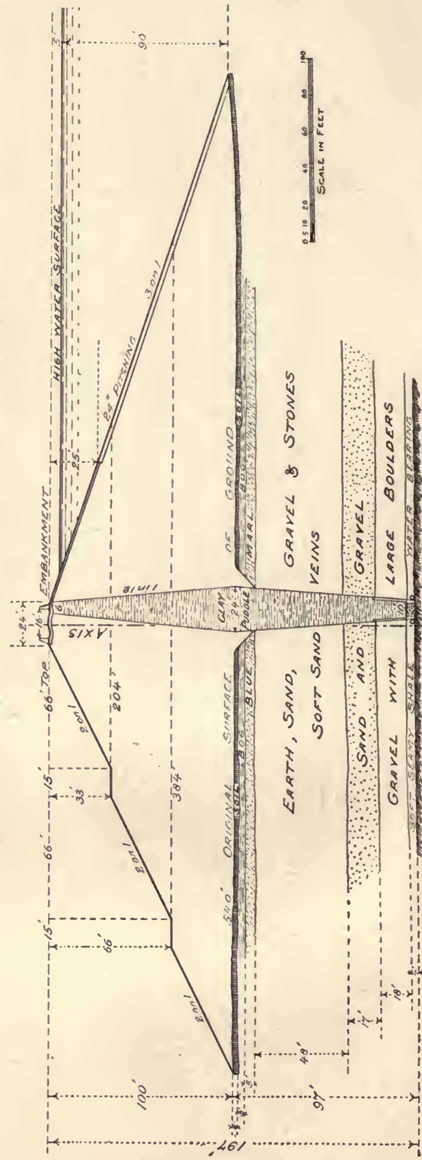


FIG. 2.—CROSS-SECTION OF YARROW DAM.

At the Yarrow dam site it was necessary to go 97 ft. below the original surface to obtain a satisfactory formation or one that was impermeable. A central trench was excavated to bed rock, parallel to the axis of the dam, and filled with clay puddle to form a water-tight connection with the rock, and prevent the water in the reservoir from passing through the porous materials under the body of the embankment. This interesting dam will be more fully described later, when the different types of earth dams are discussed.

CHAPTER III.

Outline Study of Soils. Puddle.

The following study of soils is merely suggestive and is here given to emphasize the importance of the subject, at the risk of being considered a digression. Soil formations are made in one of three ways:

1. By decomposition of exposed rocks.
2. By transportation or sedimentation of fine and coarse materials worn from rocks.
3. By transformation into humus of decayed organic matter.

The transforming agencies by which soils succeed rocks in geological progression have been classified as follows:

1. Changes of temperature.
2. Water.
3. Air.
4. Organic life.

Heat and its counter agent frost are the most powerful forces in nature, their sensible physical effects being the expansion and contraction of matter.

Water has two modes of action, physical and chemical. This agent is the great destroyer of the important forces, cohesion and friction. *Cohesion* is a force uniting particles of matter and resists their separation when the motion attempted is perpendicular to the plane of contact. *Friction* is a force resisting the separation of surfaces when motion is attempted which produces sliding. The hydrostatic pressure and resultant effect upon submerged surfaces need to be kept constantly in mind. When the surface is impermeable the line of pressure is normal to its plane, but when once saturated there are also horizontal and vertical lines of pressure. Since the strength of an earth dam depends upon two factors, namely, its weight and frictional resistance to sliding, the effect of water upon different materials entering into an earth structure should be most carefully considered. This will therefore occupy a large place in these pages. An earth embankment founded upon rock may become saturated by water forced up into it from below through cracks and fissures, reducing its lower stratum to a state

of muddy sludge, on which the upper part, however sound in itself, would slide. The best preliminary step to take in such a case is to intersect the whole site with wide, dry, stone drains, their depths varying according to the nature of the ground or rock.

Air contains two ingredients ever active in the process of decomposition, carbonic acid and oxygen.

Organic Life accomplishes its decomposing effect both by physical and chemical means. The effect of organic matter upon the mineral ingredients of the soil may be stated as follows:

1. By their hydroscopic properties they keep the soil moist.

2. Their decomposition yields carbonic acid gas.

3. The acids produced disintegrate the mineral constituents, reducing insoluble matter to soluble plant food.

4. Nitric acid results in *nitrates*, which are the most valuable form of nutritive nitrogen, while ammonia and the other salts that are formed are themselves direct food for plants.

Vegetable Humus is not the end of decomposition of organic matter, but an intermediate state of transformation. Decay is a process almost identical with combustion, where the products are the same, and the end is the formation of water and carbonic acid, with a residue of mineral ash. The conditions essential to organic decomposition are also those most favorable to combustion or oxidation, being (1) access of air, (2) presence of moisture, and (3) application of heat.

Now the coöperation of these chemical and physical forces, which are ever active, is called "weathering." Slate rock, for instance, weathers to clay, being impregnated with particles of mica, quartz, chlorite and hornblend. Shales also weather to clay, resulting often in a type of earth which is little more than silicate of aluminum with iron oxide and sand.

In the vicinity of the Tabeaud Dam, recently built under the personal supervision of the author, the construction of which will be described later, there is to be found a species of potash mica, which in decomposing yields a yellow clay (being ochre-colored from the presence of iron), mixed with particles of undecomposed mica. This material is subject to expansion, and by reason of its lack of grit and its unctuous character it was rejected or used very sparingly. Analysis of this material gave, Silica, 54.1 to 59.5%; potash, 1.5 to 2.3%; soda, 2.7 to 3.7%.

Soil analysis may be either mechanical or chemical. For purposes of earthwork, we are most interested in the former, having to

deal with the physical properties of matter. Chemical analysis, however, will often afford information of great value regarding certain materials entering into the construction of earth dams. The most important physical properties are:

(1) Weight and specific gravity.
(2) Coefficient of friction and angle of repose.
(3) Structure and coloring ingredients.
(4) Behavior toward water.

There are two distinct methods of mechanical analysis: (1) Granulating with sieves, having round holes. (2) Elutriating with water, the process being known as silt analysis.

It would require a large volume to present the subject of soil analysis in any way commensurate with its importance. Experiments bearing upon the subjects of imbibition, permeability, capillarity, absorption and evaporation, of different earth materials, are equally interesting and important.*

The permeability of soils will be discussed incidentally in connection with certain infiltration experiments to be given later.

Puddle.

Puddle without qualification may be defined as clayey and gravelly earth thoroughly wetted and mixed, having a consistency of stiff mud or mortar. Puddle in which the predominating ingredient of the mixture is pure clay, is called *clay puddle. Gravel puddle* contains a much higher percentage of grit and gravel than the last-named and yet is supposed to have enough clayey material to bind the matrix together and to fill all the voids in the gravel.

The term *earthen concrete* may also be applied to this class of material, especially when only a small quantity of water is used in the mixture. These different kinds of puddling materials may be found in natural deposits ready for use, only requiring the addition of the proper amount of water. It is usually necessary, however, to mix, artificially, or combine the different ingredients in order to obtain the right proportions. Some engineers think grinding in a pug-mill absolutely essential to obtain satisfactory results.

Puddle is handled very much as cement concrete, which is so

*The writer had intended to present a table of physical properties of different materials, giving their specific gravity, weight, coefficient of friction, angle of repose, percentage of imbibition, percentage of voids, etc., but found it impossible to harmonize the various classifications of materials given by different authorities.

well understood that detailed description is hardly necessary. Instead of tampers, sharp cutting implements are usually employed in putting puddle into place. Trampling with hoofed animals is frequently resorted to, both for the purpose of mixing and compacting.

As has been stated, clays come from the decomposition of crystaline rocks. The purest clay known (kaolin) is composed of alumina, silica and water. The smaller the proportion of silica the more water it will absorb and retain. Dry clay will absorb nearly one-third of its weight of water, and clay in a naturally moist condition 1-6 to 1-8 its weight of water. The eminent English engineers, Baker and Latham, put the percentage of absorption by clayey soils as high as 40 to 60%. Pure clays shrink about 5% in drying, while a mixture by weight of 1 clay to 2 sand will shrink about 3%. It follows, then, that the larger the percentage of clay there may be in a mixture the greater will be both the expansion and the contraction.

Clay materials may be very deceptive in some of their physical properties, being hard to pick under certain conditions, and yet when exposed to air and water will rapidly disintegrate. Beds of clay, marl and very fine sand are liable to slip when saturated, becoming semi-fluid in their nature, and will run like cream.

The cohesive and frictional resistances of clays becoming thus very much reduced when charged with water, a too liberal use of this material is to be deprecated. The ultimate particles forming clays, viewed under the microscope, are seen to be flat and scale-like, while those of sands are more cubical and spherical. This is a mechanical difference which ought to be apparent to even a superficial observer and yet has escaped recognition by many who have vainly attempted a definition of *quicksand*.

Mr. Strange recommends filling the puddle trench with material having three parts soil and two parts sand. After the first layer next to bed rock foundation, which he kneads and compacts, he would put the layers in dry, then water and work it by treading, finally covering to avoid its drying out and cracking.

Prof. Philipp Forchheimer, of Gratz, Austria, one of the highest authorities and experimentalists, affirms that if a sandy soil contains clay to such an extent that the clay fills up the interstices between the grains of sand entirely the compound is practically impervious.

Mr. Herbert M. Wilson, C. E., in his "Manual of Irrigation En-

gineering," recommends the following as an ideal mixture of materials:

	Cu. yds.		Cu. yds.
Coarse gravel	1.00	Clay	0.20
Fine gravel	0.35		——
Sand	0.15	Total	1.70

This mixture, when rolled and compacted, should give 1.25 cu. yds. in bulk, thus resulting in 26½% compression.

Mr. Clemens Herschel suggests the following test of "good binding gravel:" "Mix with water in a pail to the consistency of moist earth; if on turning the pail upside down the gravel remains in the pail it is fit for use, otherwise it is to be rejected." For *puddling material* he would use such a proportion as will render the water invisible.

CHAPTER IV.

The Tabeaud Dam, California.

The Tabeaud Dam, in Amador County, Cal., built under the supervision of the author for the Standard Electric Co., is an example of the homogeneous earth dam. A somewhat fuller description and discussion will be given of this dam than of any other, not on account of its greater importance or interest, but because it ex-

FIG. 3.—PLAN OF TABEAUD RESERVOIR, WITH CONTOURS.

emplifies certain principles of construction upon which it is desired to put special emphasis. This dam was described in Engineering News of July 10, 1902, to which the reader is referred for more complete information than is given here.

Fig. 3 is a contour map of the Tabeaud Reservoir, showing the relative locations of the dam, wasteway and outlet tunnel. Fig. 4 shows the bed rock drainage system and the letters upon the draw-

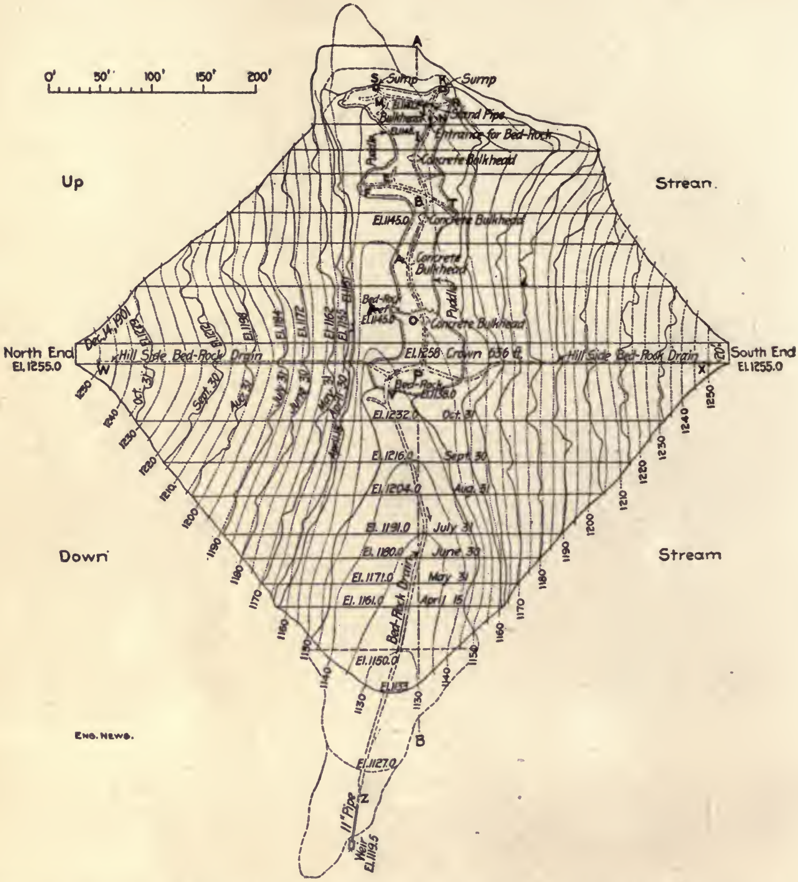


FIG. 4.—PLAN OF TABEAUD DAM, SHOWING BED ROCK DRAINAGE SYSTEM.

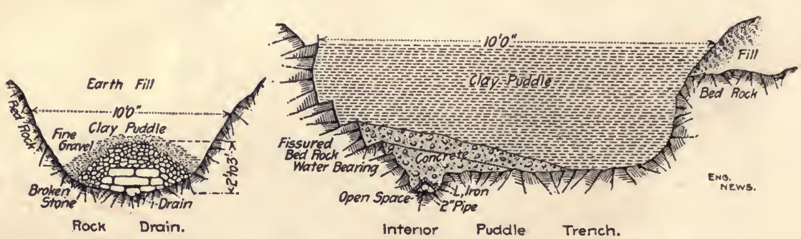


FIG. 5.—DETAILS OF BED ROCK DRAINS AT THE TABEAUD DAM.

ing will assist in following the explanation given in the text. The whole up-stream half of the dam site was stripped to bed rock. As the work of excavation advanced pockets of loose alluvial soil were encountered, which were suggestive of a refill, possibly the result of placer mining operations during the early mining days of California. In addition to this were found thin strata ot sand and gravel deposited in an unconformable manner. The slate bed rock near the up-stream toe of the dam was badly fissured and yielded considerable water. A quartz vein from 1 to 2 ft. in thickness crossed the dam site about 150 ft. above the axis of the dam. The slate rock

FIG. 6.—VIEW OF BED ROCK TRENCHES, TABEAUD DAM.

above this vein or fault line was quite variable in hardness and dipped at an angle of 40 degrees toward the reservoir.

The rear drain terminates at a weir box (Z) outside of the down-stream slope at a distance of 500 ft. from the axis of the dam. This drain branches at the down-stream side of the central trench, (Y), one branch being carried up the hillside to high-water level (W) at the North end of the dam, and the other to the same elevation at the South end (X).

Fig. 5 shows how these drains were constructed. After the removal of all surface soil and loose rock, a trench 5 to 10 ft. wide was cut into the solid rock, the depth of cutting varying with the character of the bed rock. Upon the floor of this trench a small open drain was made by notching the bed rock and by means of selected stones of suitable size and hardness. The stringers and cap-stones were carefully selected and laid, so that no undue settlement or displacement might occur by reason of the superincumbent

FIG. 7.—VIEW OF NORTH TRENCH, TABEAUD DAM.

weight of the dam. All crevices were carefully filled with spawls and the whole overlaid 18 ins. in depth with broken stone 1 to 3 ins. in diameter. Upon this layer of broken stone and fine gravel was deposited choice clay puddle, thoroughly wetted and compacted, refilling the trenches.

These drains served a useful purpose during construction, in drying off the surface of the dam after rains. The saturation of the outer slope of the dam by water creeping along the line of contact

FIG. 8.—VIEW OF SOUTH TRENCH, TABEAUD DAM.

FIG. 9.—VIEW OF MAIN CENTRAL DRAIN, TABEAUD DAM.

should thus be prevented, and the integrity or freedom from saturation of the down-stream half should be preserved. It is believed that the puddle overlying these rock drains will effectually prevent any water from entering the body of the embankment by upward pressure and that the drains will thus forever act as efficient safeguards.

The main drain was extended, temporarily during construction, from the central trench (Fig. 4), to the up-stream toe of the dam. This was cut 5 or 6 ft. deep into solid rock, below the general level of the stripped surface. Fig. 6 is reproduced from a photograph of this trench. An iron pipe 2 ins. in diameter was imbedded in Portland cement mortar and concrete, and laid near the bottom of the trench.

At the point (B) where the quartz vein (already described) intersected this drain, two branch drains were made, following the fault well into the hill on both sides. Figs. 7 and 8 are views of the North and South trenches, respectively. These trenches were necessary to take care of the springs issuing along the quartz vein. This water led to a point (N, Fig. 4) near the up-stream toe, by means of the drain shown in Fig. 9.

The lateral drains and that portion of the main central drain extending from their junction (B) to a point (N) about 230 ft. from the axis of the dam have pieces of angle iron or wooden Y-fluming laid on the bottom of the trenches immediately over the 2-in. pipe, as shown in Figs. 7, 8 and 9. These are covered in turn with Portland cement mortar, concrete, clay puddle and earth fill. The water will naturally flow along the line of least resistance, and consequently will follow along the open space between the angle irons and the outside of the pipe until it reaches the chamber and opening in the pipe, permitting the water to enter and be conveyed through the imbedded pipe line to the rear drain. This point of entry is a small chamber in a solid cross-wall of rich cement mortar, and is the only point where water can enter this pipe line, the two branches entering the wells and the stand-pipe at their junction (soon to be described) having been closed.

That portion of the foundation between the axis of the dam and the quartz vein, a distance of about 160 ft., was very satisfactory, without fissures or springs of water. In this portion the 2-in. pipe was imbedded in mortar and concrete without angle irons, and the continuity of the trench broken by numerous cross-trenches cut into the rock and filled with concrete and puddle. It

FIG. 10—VIEW OF TABEAUD DAM WHEN ABOUT HALF COMPLETED.

is believed that no seepage water will ever pass through this portion of the dam. If any should ever find its way under the puddle and through the bed rock formation, the rear drain, with its hillside branches, will carry it away and prevent the saturation of the lower or down-stream half of the dam.

At the up-stream toe of the embankment, two wells or sumps (shown at "S" and "K," Fig. 4) were cut 10 or 12 ft. deeper than the main trench, which received the water entering the inner toe puddle trench during construction. This water was disposed of partly by pumping and partly by means of the 2-in. branch pipes leading into and from these wells. At their junction (J) a 2-in. stand-pipe was erected, which was carried vertically up through the embankment, and finally filled with cement. The branch pipes from the wells were finally capped and the wells filled with broken stone, as previously mentioned.

EMBANKMENT.—As has been said, the upper surface of the slate bed rock was found to be badly fissured, especially near the upstream toe of the dam, and as the average depth below the surface of the ground was not very great, it was thought best to lay bare the bed rock over the entire upper half of the dam site. Had the depth been much greater, it would have been more economical and possibly sufficient to have put reliance in a puddle trench, alone, for securing a water-tight connection between the foundation and the body of the dam.

At the axis of the dam and near the inner toe, where the puddle walls abutted against the hillsides, the excavation always extended to bed rock. Vertical steps and offsets were avoided and the cuts were made large enough for horses to turn in while tramping, these animals being used, singly and in groups, to mix and compact the puddle and thus lessen the labor of tamping by hand. In plan, the hillside contact of natural and artificial surfaces presents a series of corrugated lines, (as is clearly shown in Fig. 4.) After all loose and porous materials had been removed, the stripped surface and the slopes of all excavations were thoroughly wetted from time to time by means of hose and nozzle, the water being delivered under pressure. Fig. 10 is a view of the dam taken when it was about half finished and shows the work in progress.

The face puddle shown in Fig. 11 was used merely to "make assurance doubly sure" and was not carried entirely up to the top of the dam. The earth of which the dam was constructed may

be described as a red gravelly clay, and in the judgment of the author is almost ideal material for the purpose. Physical tests and experiments made with the materials at different times during construction gave the following average results:

	Pounds.
Weight of 1 cu. ft. earth, dust dry	84.0
" " 1 " saturated earth	101.8
" " 1 " moist loose earth	76.6
" " 1 " loose material taken from test pits on the dam	80.0
" " 1 " earth in place taken from the borrow pits	116.5
" " 1 " earth material taken from test-pits on the dam.	133.0

	Per cent.
Percentage of moisture in natural earth	19
" " voids in natural earth	52
" " grit and gravel in natural earth	38
" " compression on dam over earth at borrow-pit	16
" " compression on dam over earth in wagons	43

	Degrees.
Angle of repose of natural moist earth	44
Angle of repose of earth, dust dry	36
Angle of repose of saturated earth	23

CONSTRUCTION DETAILS.—The materials forming the bulk of the dam were hauled by four-horse teams, in dump-wagons, holding 3 cu. yds. each. The wagons loaded weighed about six tons and were provided with two swinging bottom-doors, which the driver could operate with a lever, enabling the load to be quickly dropped while the team was in motion. If the material was quite dry, the load could be dumped in a long row when so desired.

After plowing the surface of the ground and wasting any objectionable surface soil, the material was brought to common earth-traps for loading into wagons, by buck or dragscrapers of the Fresno pattern. In good material one trap with eight Fresno-scraper teams could fill 25 wagons per hour. The average length of haul for the entire work was about 1,320 ft.

The original plans and specifications were adhered to throughout, with the single exception that the central puddle wall was not carried above elevation 1,160, as shown on Figs. 11 and 12, more attention being given to the inner face puddle. This modification in the original plans was made because of the character of the materials available and the excellent results obtained in securing an homogeneous earthen-concrete, practically impervious.

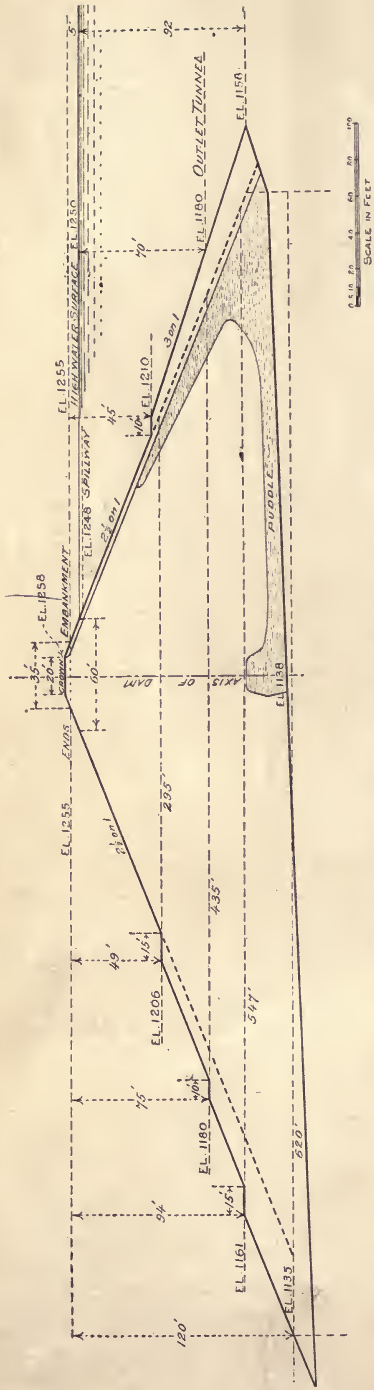


FIG. 11.—DIMENSION SECTION OF TABEAUD DAM.

The top of the embankment was maintained basin-shaped during construction, being lower at the axis than at the outer slopes by 1-10, to the height below the finished crown. This gave a grade of about 1 in 25 from the edges toward the center, resulting in the following advantages:

(1) Insuring a more thorough wetting of the central portion of the dam; any excess of water in this part would be readily taken care of by the central cross drains.

(2) In wetting the finished surface prior to depositing a new layer of material, water from the sprinkling wagons would naturally drain towards the center and insure keeping the surface wet; the layers being carried, as a rule, progressively outward from the center.

(3) It centralized the maximum earth pressure and enabled the depositing of material in layers perpendicular to the slopes.

(4) It facilitated rolling and hauling on lines parallel to the axis of the dam, and discouraged transverse and miscellaneous operations.

(5) It finally insured better compacting by the tramping of teams in their exertions to overcome the grade.

The specifications stipulated that the body of the dam should

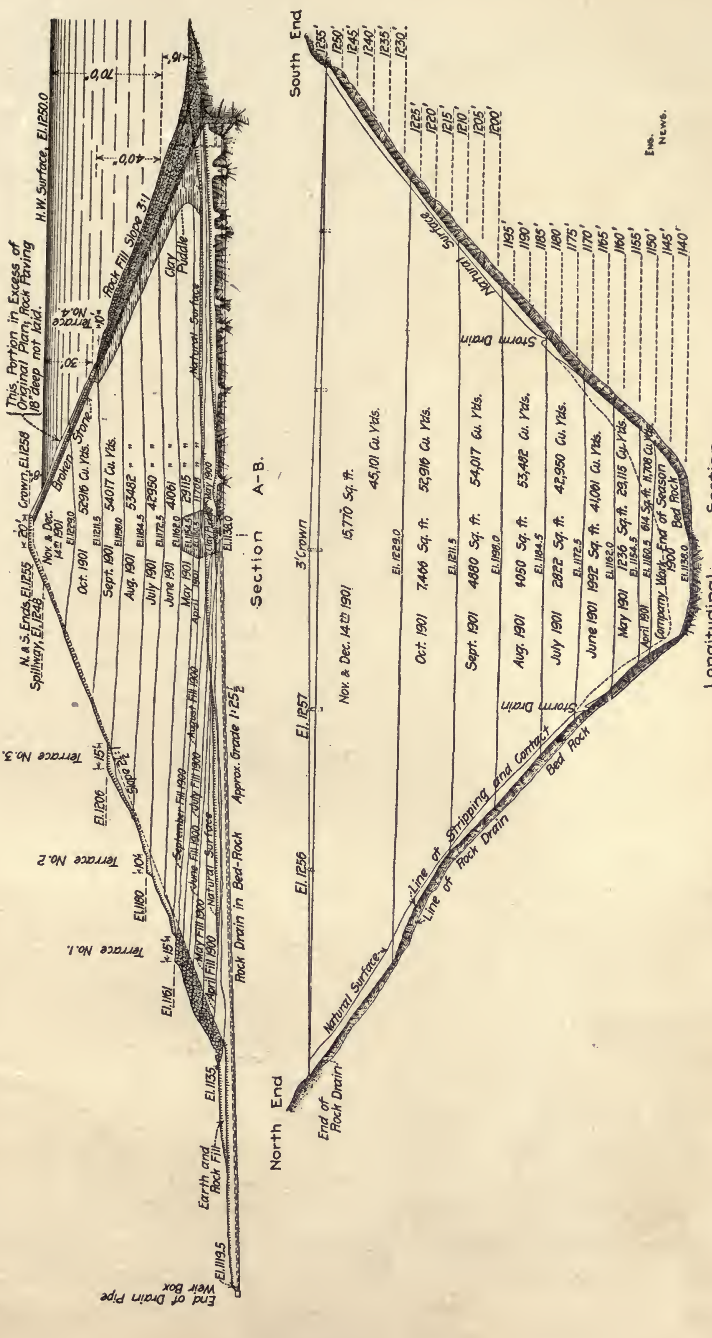


FIG. 12.—CROSS AND LONGITUDINAL SECTIONS OF TABEAUD DAM.

be built up in layers not exceeding 6 ins. in thickness for the first 60 ft., and not exceeding 8 ins. above that elevation. The finished layers after rolling varied slightly in thickness, the daily average per month being as follows:

April	4 ins.	August	5 ins.
May	$3\frac{1}{2}$"	September	6 "
June	4 "	October	7 "
July	$4\frac{1}{2}$"	November and December	8 "

During the last few months more than one whole layer constituted the day's work, so that a single layer was seldom as thick as the daily average indicates.

It was stipulated in the specifications that the up-stream half of the dam was to be made of "selected material" and the lower half of less choice material, not designated "waste." "Waste material" was described as meaning all vegetable humus, light soil, roots, and rock exceeding 5 lbs. in weight, too large to pass through a 4-in. ring.

It may be well to define the expression "selected material," so commonly used in specifications for earth dams. In England, for instance, it is said to refer to materials which insure *water-tightness*, while in India it refers to those employed to obtain *stability*. It ought to mean the best material available, selected by the engineer to suit the requirements of the situation.

The method employed in building the body of the embankment may be described as follows:

(1) The top surface of every finished layer of material was sprinkled and harrowed prior to putting on a new layer. The sprinkling wagons passed over the older finished surface immediately before each wagon-row was begun. This insured a wetted surface and assisted the wheels of the loaded wagons, as well as the harrows, to roughen the old surface prior to depositing a new layer.

(2) The material was generally deposited in rows parallel to the axis of the dam. However, along the line of contact, at the margins of the embankment, the earth was often deposited in rows crosswise of the dam, permitting a selection of the choicest materials and greatly facilitating the work of graders and rollers.

(3) Rock pickers with their carts were continually passing along the rows gathering up all roots, rocks and other waste materials.

(4) The road-graders drawn by six horses leveled down the tops of the wagon-loads, and if the material was dry the sprinkling

FIG. 13.—VIEW OF TABEAUD DAM IMMEDIATELY AFTER COMPLETION.

wagons immediately passed over the rows prior to further grading. When the material was naturally moist the grader continued the leveling process until the earth was evenly spread. The depth or thickness of the layer could be regulated to a nicety by properly spacing the rows and the individual loads. The grader brought the layer to a smooth surface and of uniform thickness, and nothing more could be desired for this operation.

(5) After the graders had finished, the harrows passed over the new layer to insure the picking out of all roots and rocks, followed immediately by the sprinkling wagons.

(6) Finally the rollers thoroughly compacted the layer of earth, generally passing to and fro over it lengthwise of the dam. Along the line of contact at the ends, however, they passed crosswise. Then again they frequently went around a portion of the surface until the whole was hard and solid.

Two rollers were in use constantly, each drawn by six horses. One weighed five tons and the other eight tons, giving respectively 166 and 200 lbs. pressure per lin. in. They were not grooved, but the smooth surface left by the rollers was always harrowed and cut up more or less by the loaded wagons passing over the surface previously wetted. The wagons when loaded gave 750 lbs. pressure per lin. in., and the heavy teams traveling wherever they could do the most effective work compacted the materials better even than the rollers.

Several test pits which were dug into the dam during construction showed that there were no distinct lines traceable between the layers and no loose or dry spots, but that the whole mass was solid and homogeneous.

A careful record is being kept of the amount of settlement of the Tabeaud Dam. It will be of interest to record here the fact that just one year after date of completion the settlement amounted to 0.2 ft., with 90 ft. depth of water in the reservoir.

Water was first turned into the reservoir five months after the dam was finished. The very small amount of settlement here shown emphasizes more eloquently than words the author's concluding remarks relating to the importance of thorough consolidation, by artificial means, of the embankment. (See p. 64, Secs. 6 to 8.)

OUTLET TUNNEL.—The outlet for the reservoir is a tunnel 2,903 ft. in length, through a ridge of solid slate rock formation, which was very hard and refractory. At the north or res-

ervoir end of the tunnel, there is an open cut 350 ft. long, with a maximum depth of 26 ft.

Near the south portal of the tunnel and in the line of pressure pipes connecting the "petty reservoir" above with the power-house below, is placed a receiver, connected with the tunnel by means of a short pipe-line, 60 ins. in diameter.

A water-tight bulkhead of brick and concrete masonry is placed in the tunnel, at a point about 175 ft. distant from the receiver. In the line of 60-in. riveted steel pipe, which connects the reservoir and tunnel with the receiver, there is placed a cast iron chamber for entrapping silt or sand, with a branch pipe 16 ins. in diameter leading into a side ravine through which sand or silt thus collected can be wasted or washed out. By the design of construction thus described, it will be seen that all controlling devices, screens, gates, etc., are at the south end of the tunnel and easily accessible.

WASTEWAY.—The wasteway for the reservoir is an open cut through its rim, 48 ft. in width and 300 ft. long. The sill of the spillway is 10 ft. below the crown of the dam. The reservoir having less than two square miles of catchment area, and the feeding canals being under complete control, the dam can never be overtopped by a flood. Fig. 3 shows the relative location of the dam, outlet tunnel and wasteway channel.

Almost the whole of the embankment forming the Tabeaud Dam, not included in the foundation work, was built in less than eight months. The contractor's outfit was the best for the purpose the writer has ever seen. After increasing his force from time to time he finally had the following equipment:

1	steam shovel (1½ yds. capacity),
37	patent dump wagons,
11	stick-wagons and rock-carts,
39	buck-scrapers (Fresno pattern),
21	wheel scrapers,
3	road-graders,
3	sprinkling-wagons,
2	harrows,
2	rollers (5 and 8-ton),
233	men,
416	horses and mules.
8	road and hillside plows,

STATISTICS.—The following data relating to the Tabeaud Dam Reservoir will conclude this description:

DAM.

Length at crown	636 ft.
Length at base crossing ravine	50 to 100 "
Height to top of crown (El. 1,258.)	120 "
" at ends above bedrock	117 "
" at up-stream toe	100 "
" at down-stream toe	123 "
Effective head	115 "
Width at crown	20 "
Width at base	620 "

Slopes, 2½ on 1 with rock-fill 3 to 1.

Excavation for foundations	40,000 cu. yds.
Refill by company	40,000 "
Embankment built by contractor	330,350 "
Total volume of dam	370,350 "
Total weight	664,778 tons.
Length of wasteway (width)	48 ft.
Depth of spillwav sill below crown	10 "
Depth of spillway sill below ends	7 "
Height of stop-planks in wasteway	2 "
Maximum depth of water in reservoir	92 "
Area to be faced with stone	1,933 sq. yds.

RESERVOIR.

Catchment area (approximate)	2 sq. miles.
Area of water surface	36. 75 acres.
Silt storage capacity below outlet tunnel	1,091,470 cu. ft.
Available water storage capacity	46,612,405 "
Elevation of outlet tunnel	1,180 ft.
" " high-water surface	1,250 "
" " crown of dam	1,258 "

Fig. 13 is a view of the finished dam, taken immediately after completion.

CHAPTER V.

Different Types of Earth Dams.

There are several types of earth dams, which may be described as follows:

1. Homogeneous earth dams, either with or without a puddle trench.
2. Earth dams with a puddle core or puddle face.
3. Earth dams with a core wall of brick, rubble or concrete masonry.
4. New types, composite structures.
5. Rock-fill dams with earth inner slope.
6. Hydraulic-fill dams of earth and gravel.

The writer proposes to give an example of each type, with such remarks upon their distinctive features and relative merits as he thinks may be instructive.

Earth Dams with Puddle Core Wall or Face.

YARROW DAM.—The Yarrow dam of the Liverpool Water-Works is a notable example of the second type, (a section of which is shown in Fig. 2.) An excavation 97 ft. in depth was made to bed rock through different strata of varying thickness, and a trench 24 ft. wide was cut with side slopes 1 on 1 for the first 10 ft. in depth below the surface. The trench was then carried down through sand, gravel and boulders with sides sloping 1 in 12. The upper surface of the shale bed rock was found to be soft, seamy and water-bearing. Pumps were installed to keep the water out of the trench while it was being cut 4 or 5 ft. deeper into the shale. The lower portion was then walled up on either side with brickwork 14 ins. in thickness, and the trench between the walls was filled with concrete, made in the proportion of 1 of cement, 1 of sand and 2 of gravel or broken stone. By so doing a dry bed was secured for the foundation of the puddle wall. Two lines of 6-in. pipes were laid on the bed rock, outside of the walls, and pipes 9 ins. in diameter extended vertically above the top of the brickwork some 27 ft. These pipes were filled with concrete, after disconnecting the pumps. After refilling the trench with puddle to the original surface, a puddle wall was carried up simultan-

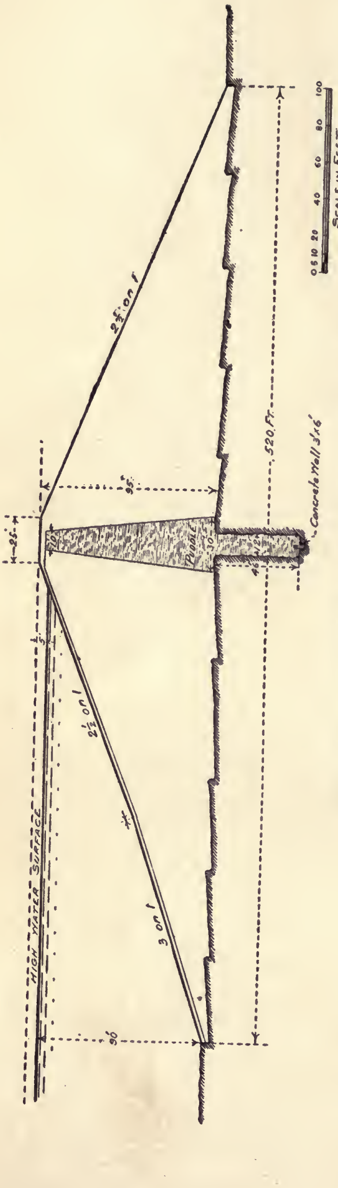


FIG. 14.—CROSS-SECTION OF PILARCITOS DAM.

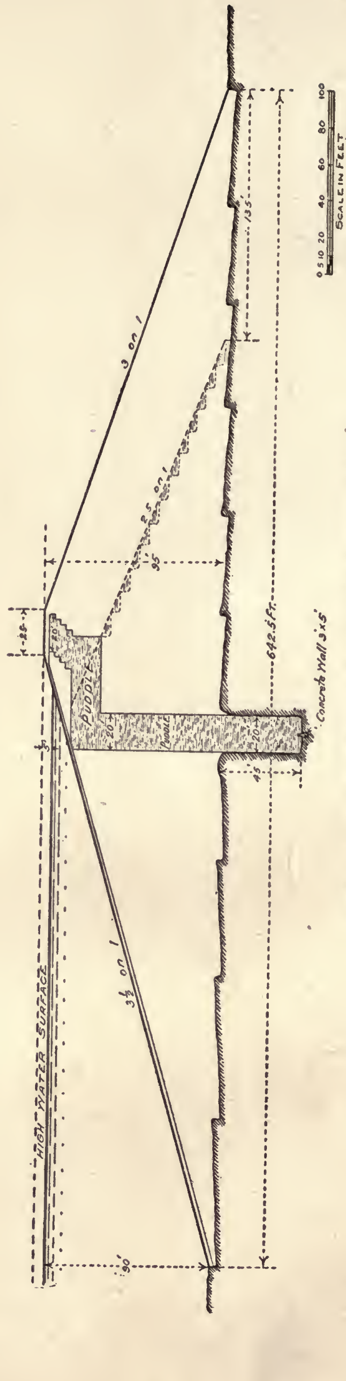


FIG. 15.—CROSS-SECTION OF SAN ANDRES DAM.

eous with the embankment, having a decreasing batter of 1 in 12, which gave a width of 6 ft. at the top. This form of construction is very common in England, and Figs. 14 and 15 show two California dams, the Pilarcitos and San Andres, of the same general type.

ASHTI EMBANKMENT.—This is not a very high embankment, but being typical of modern dams in British India, where the puddle is generally carried only to the top of the original surface of the ground, and not up through the body of the dam, it is thought worthy of mention. Fig. 16 shows a section of this embankment, which is located in the Sholapur District, India.

The central portion of this dam above the puddle trench is made of "selected black soil;" then on either side is placed "Brown Soil," finishing on the outer slopes with "Murum." Trap rock decomposes first into a friable stony material, known in India as "Murum" or "Murham." This material further decomposes into

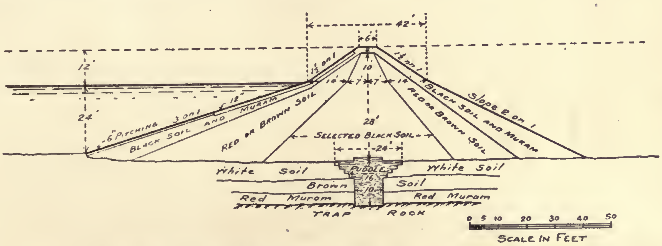


FIG. 16.—CROSS-SECTION OF ASHTI TANK EMBANKMENT.

various argillaceous earths, the most common being the "black cotton soil" mentioned above.

This particular dam has been adversely criticised on account of the lack of uniformity in the character of the materials composing the bank. It is claimed that the materials being of different density and weight, unequal settlement will result, and lines of separation will form between the different kinds of materials.

Earth materials do not unite or combine with timber or masonry, but there are no such distinct lines of transition and separation between different earth materials themselves as Fig. 16 would seem to indicate.

Puddle Trench.

In the last three dams mentioned (Figs. 14, 15, 16) the puddle trenches are made with vertical sides or vertical steps and offsets. A wedge-shaped trench certainly has many advantages over this

form. Puddle being plastic, consolidates as the dam settles, filling the lowest parts by sliding on its bed. It thus has a tendency to break away from the portion supported by the step, and a further tendency to leave the vertical side, thus forming cracks and fissures for water to enter. The argument advanced by those holding a different view, namely, that it is difficult to dress the sides of a trench to a steep batter and to timber it substantially, has in reality little weight when put to practical test. Mr. F. P. Stearns, in describing the recent work of excavating the cut-off trench of the North Dike of the Wachusett reservoir, Boston, said it was found to be both better and cheaper to excavate a trench with slopes than with vertical sides protected by sheeting. He favored this shape in case of pile-work and for the purpose also of wedging materials together.

Mr. Wm. J. McAlpine's "Specifications for Earth Dams," representing the best practice of 25 years ago, which are frequently cited, contain the following description of how to prepare the up-stream floor of the dam:

> Remove the pervious and decaying matter by breaking up the natural soil and by stepping up the sides of the ravine; also by several toothed trenches across the bottom and up the sides.

One of Mr. McAlpine's well known axioms was, "water abhors an angle." The "stepping" and "toothed" trenches above specified need not necessarily be made with vertical planes, but should be made by means of inclined and horizontal planes. The writer's experience and observation leads him to think that all excavations in connection with earth dams requiring a refill should be made wedge-shaped so that the pressure of the superincumbent materials in settling will wedge the material tighter and tighter together and fill every cavity. A paper by Mr. Wm. L. Strange, C. E., on "Reservoirs with high Earthen Dams in Western India," published in the Proceedings of the Institution of Civil Engineers, Vol. 132, (1898), is one of the best contributions to the literature on this subject, known to the writer. Mr. Strange states that

> the rate of filtration of a soil depends upon its porosity, which governs the frictional resistance to flow, and the slope and length of the filamentary channels along which the water may be considered to pass. It is evident, therefore, that the direct rate of infiltration in a homogeneous soil must decrease from the top to the bottom of the puddle trench. The best section for a puddle trench is thus a wedge, such as an open excavation would give. It is true that the uppermost infiltrating filaments when stopped by the puddle, will endeavor to get under it, but a depth will eventually be reached

when the frictional resistance along the natural passages will be greater than that due to the transverse passage of the puddle trench, and it is when this occurs that the latter may be stopped without danger, as the *filtration to it* will be less than that that *through it.* This depth requires to be determined in each case, but in fairly compact Indian soils 30 feet will be a fair limit.

Puddle Wall vs. Puddle Trench.

There is a diversity of opinion among engineers in regard to the proper place for the puddle in dam construction. Theoretically, the inner face would be preferable to the center, for the purpose of preventing any water from penetrating the embankment. It is well known that all materials immersed in water lose weight in proportion to the volume of water they displace. If the upper half of the dam becomes saturated it must necesarily lose both weight and stability. Its full cohesive strength can only be maintained by making it impervious in some way. The strength of an earth dam depends upon three factors:

1. Weight.
2. Frictional resistance against sliding.
3. Cohesiveness of its materials.

These can be known only so long as no water penetrates the body of the dam. When once saturated the resultant line of pressure is no longer normal to the inner slope, for the reason that there is now a force tending to slide the dam horizontally and another due to the hydrostatic head tending to lift it vertically. When the water slope is impervious the horizontal thrust is sustained by the whole dam and not by the lower half alone. When once a passage is made into the body of the dam, the infiltration water will escape along the line of least resistance, and if there be a fissure it may become a cavity and the cavity a breach.

For practical reasons, mainly on account of the difficulty of maintaining a puddle face on the inner slope of a dam, which would require a very flat slope, puddle is generally placed at the center as a core wall.

It was thought possible at the Tabeaud dam to counteract the tendency of the face puddle to slough off into the reservoir by use of a broken stone facing of riprap. This covering will protect the puddle from the deteriorating effects of air and sun whenever the water is drawn low and also resists the pressure at the inner toe of the dam.

Percolation and Infiltration.

The earlier authorities on the subject of percolation and infiltration of water are somewhat conflicting in their statements, if not confused in their ideas. We are again impressed with the importance of a clearly defined and definite use of terms. The temptation and tendency to use language synonymously is very great, but it is unscientific and must result in confusion of thought. Let it be observed that *filtration* is the process of mechanically separating and removing the undissolved particles floating in a liquid. That *infiltration* is the process by which water (or other liquid) enters the interstices of porous material. That *percolation* is the action of a liquid passing through small interstices; and, finally, that *seepage* is the amount of fluid which has percolated through porous materials.

Many recent authorities are guilty of confusion in thought or expression, as will apoear from the following:

One says, for instance, that a

rock is water-tight when non-absorbent of water, but that a soil is not water-tight unless it will absorb an enormous quantity of water.

This would seem to indicate that super-saturation and not pressure is necesary to increase the water-tightness of earth materials.

Again, in a recent discussion regarding the saturation and percolation of water through the lower half of a reservoir embankment, it was remarked, that

the more compact the material of which the bank is built, the steeper will be the slope of saturation.

Exception was taken to this, and the statement made, that *with compact material,* the sectional area of flow is larger below a given level with porous material, and as the bank slope is one determining factor of the line of saturation, this line tends to approach the slope line; while with porous material in a down-stream bank, the slope of saturation is steeper and the area of the flow less.

In reply to this, it was said,

that it is obvious that if the embankment below the core wall is built of material so compact as to be impervious to water, no water passing through the wall will enter it, and the slope of saturation will be vertical. If it be less compact, water will enter more or less according to the head or pressure, and according to its compactness or porosity, producing a slope of saturation whose inclination is dependent on the frictional resistance encountered by the water. And the bank will be tight whenever the slope of saturation remains within the figure of the embankment.

Further,

that it was necessary to distinguish between the slope assumed by water *retained in* an embankment and that taken by water *passing through* an embankment made of material too porous to retain it; where the rule is clearly reversed and where the more porous the material the steeper the slope at which water will run through it at a given rate.

These citations are sufficient to emphasize the importance of exact definition of terms and clear statement of principles.

The latest experiments relating to the percolation of water through earth materials and tests determining the stability of soils are those made during the investigations at the New Croton Dam and Jerome Park Reservoir, New York, and those relating to the North Dike of the Wachusett Reservoir, Boston. These are very interesting and instructive, and it is here proposed to discuss the results and conclusions reached in these cases, after some introductory remarks reciting the order of events.

NEW CROTON DAM.—In June, 1901, the Board of Croton Aqueduct Commissioners of New York requested a board of expert engineers, consisting of Messrs. J. J. R. Croes, E. F. Smith and E. Sweet, to examine the plans for the construction of the earth portion of the New Croton Dam, and also the core wall and embankment of the Jerome Park reservoir.

This report was published in full in Engineering News for Nov. 28, 1901. It was followed in subsequent issues of the said journal by supplemental and individual reports from each member of the board of experts, and by articles from Messrs. A. Fteley, who originally designed the works, A. Craven, formerly division engineer on this work, and W. R. Hill, at that time chief engineer of the Croton Aqueduct Commission.

After describing the New Croton Dam, the board of experts preface their remarks on the earth embankment by saying that

it has been abundantly proven that up to a height of 60 or 70 ft. an embankment founded on solid material and constructed of well-selected earth, properly put in place, is fully as durable and safe as a masonry wall and far less costly.

There are, in fact, no less than 22 earth dams in use to-day exceeding 90 ft. in height, and twice that number over 70 ft. in height. Five of the former are in California, and several of these have been in use over 25 years. The writer fails to appreciate the reason for limiting the safe height of earth dams to 60 or 70 ft.

The New Croton Dam was designed as a composite structure of masonry and earth, crossing the Croton Valley at a point three

miles from the Hudson River. The earth portion was to join the masonry portion at a point where the latter was 195 ft. high from the bed rock. The Board thought there was no precedent for such a design and no necessity for this form of construction. The point to be considered here was whether a dam like this can be made sufficiently impermeable to water to prevent the outer slope from becoming saturated and thus liable to slide and be washed out.

The design of the embankment portion was similar to all the earth dams of the Croton Valley. In the center is built a wall of rubble masonry, generally founded upon solid rock, and "intended to prevent the free seepage of water, but not heavy enough to act alone as a retaining wall for either water or earth."

Fig. 17 shows a section which is typical of most New England earth dams; and Fig. 18, the sections of two of the Croton Val-

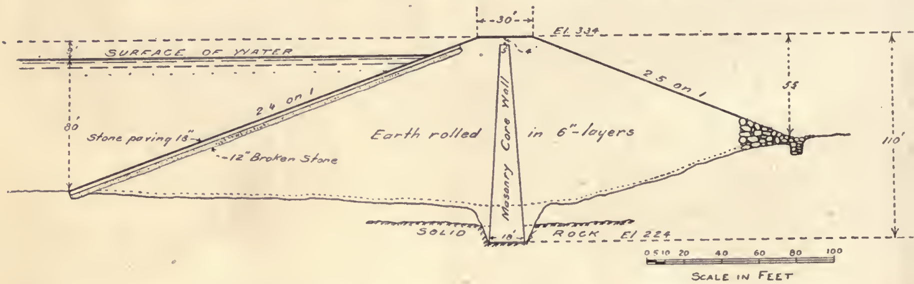


FIG. 17.—CROSS-SECTION OF A TYPICAL NEW ENGLAND DAM.

ley dams, New York water supply. These dams all have masonry core walls, illustrating the third type of dams given on page 33.

The board of experts made numerous tests by means of borings into the Croton Valley dams to determine the slope of saturation. The hydraulic laboratory of Cornell University also made tests of the permeability of several samples of materials taken from pits. All the materials examined were found to be permeable and when exposed to water to disintegrate and assume a flat slope, the surface of which was described as "slimy."

Pipe wells were driven at different places into the dams and the line of saturation was determined by noting the elevations at which the water stood in them. In all the dams the entire bank on the water side of the core wall appeared to be completely saturated. Water was also found to be standing in the embankment on the down-stream side of the core wall. The extent of saturation of the outer bank varied greatly, due to the difference in materials, the

care taken in building them, and their ages. Fig. 19 gives the average slopes of saturation as determined by these borings.

The experts stated

> that the slope of the surface of the saturation in the bank is determined by the solidity of the embankment: The more compact the material of which the bank is built, the steeper will be the slope of saturation.

As a result of their investigations, the experts were of the opinion that the slope of saturation in the best embankments made of the material found in the Croton Valley is about 35 ft. per 100 ft., and that with materials less carefully selected and placed the slope may be 20 ft. per 100 ft.

Further, that taking the loss of head in passing through the core wall, and the slope assumed by the plane of saturation, the maximum safe height of an earth dam with its top 20 ft. above water level in the reservoir and its outside slope 2 on 1, is 63 to 102.5 ft. This is a remarkable finding in view of the fact that the Titicus Dam, one of the Croton Valley dams examined, has a maximum height above bed rock of 110 ft. and has been in use seven years. This dam is not a fair example to cite in proof of their conclusion, because its *effective head* is only about 46 ft.*

Mr. Fteley gave as a reason for the elevation of the water slope

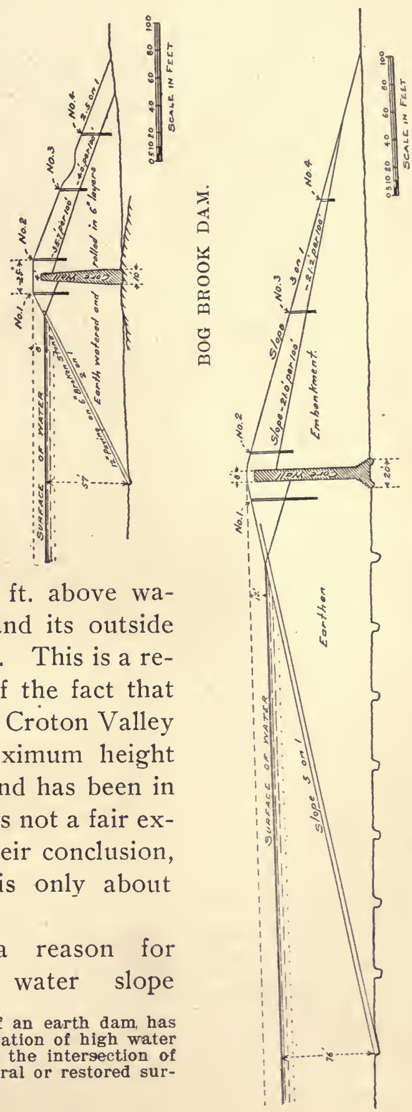


FIG. 18.—CROSS-SECTIONS OF TWO CROTON VALLEY DAMS, SHOWING SATURATION.

*The effective head at any point of an earth dam, has been defined as the difference in elevation of high water surface in the reservoir and that of the intersection of the down-stream slope with the natural or restored surface of the ground below the dam.

found in the outer bank of the Croton dams the fact of their being constructed of fine materials and stated that with comparatively porous materials they would have shown steeper slopes of saturation.

Mr. Craven argued that all dams will absorb more or less water, and that porosity is merely a degree of compactness; that slope implies motion in water, and that there is no absolute retention of water in the outer bank of a dam having its base below the plane indicated by the loss of head in passing through the inner bank and then through a further obstruction of either masonry or puddle; that there is simply a partial retention, with motion through the bank governed by the degree of porosity of the material.

Fig. 19 is a graphical interpretation of the conclusion reached by the board of experts, as already given on page 41. "A" is an ideal profile of a homogeneous dam with the inner slope 3 on 1 and the outer slope 2 on 1. The top width is made 25 ft. for a dam having 90 ft. effective head, the high water surface in the reservoir being 10 ft. below the crest of the dam. This ideal profile is a fair average of all the earth dams of the world. Not having a core wall to augment the loss of head, it fairly represents what might be expected of such a dam built of Croton Valley material, compacted in the usual way. It should be noted that the intersection of the plane of saturation with the rear slope of the dam at such high elevation as shown indicates an excessive seepage and a dangerously unstable condition.

Preliminary Study of Profile for Dam.

The preliminary calculations for desiging a profile for an earth dam are simple and will here be illustrated by an example. Let us assume the following values:

a. Central height of dam, 100 ft.

b. Maximum depth of water, 90 ft., with surface 10 ft. below crest of dam.

c. Effective head, 90 ft.

d. Weight of water, 62.5 lbs. per cu. ft.

e. Weight of material, 125 lbs. per cu. ft.

f. Coefficient of friction, 1.00, or equal to the weight.

g. Factor of safety against sliding, 10.

The width corresponding to the vertical pressure of 1 ft. is,

$$\frac{62.5 \times 10}{125} = 5 \text{ ft.}$$

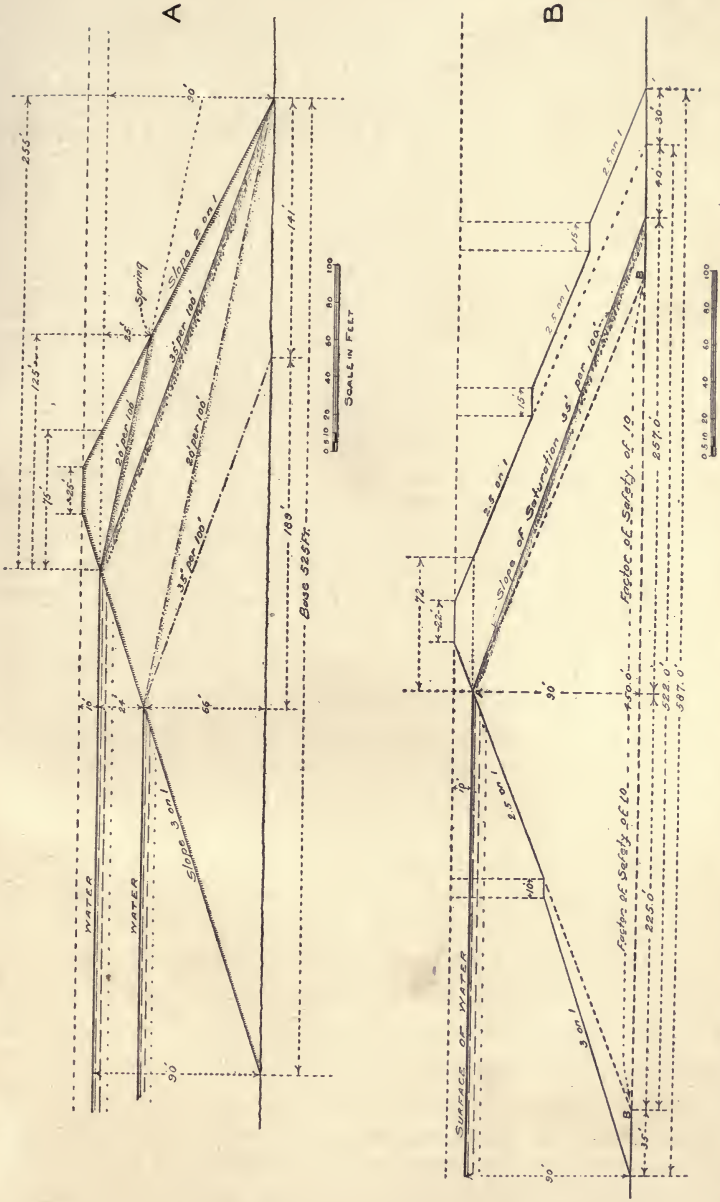


FIG. 19.—GRAPHICAL INTERPRETATION OF STUDIES OF BOARD OF EXPERTS ON THE ORIGINAL EARTH PORTION OF THE NEW CROTON DAM.

The hydrostatic pressure per square foot at 90 ft. depth is, $62.5 \times 90 = 5{,}625$ lbs.

The dam, having a factor of safety of 10, must present a resistance of, $5{,}625 \times 10 = 56{,}250$ lbs., or 28 tons per square foot.

The theoretical width of bank corresponding to 90 ft. head and a factor of 10 is shown by the dotted triangle (A-B-B) to be 450 ft., (B, Fig. 19) with slopes $2\frac{1}{2}$ on 1.

To this must be added the width due to the height of crest above the water surface in the reservoir and the width of crest.

The former would be, $2\,(2\frac{1}{2} \times 10) = 50$ ft., and the latter by Trautwine's rule, $2 + 2\sqrt{100} = 22$ ft., giving a total base width of 522 ft.

Let us now assume that the slope of saturation may be 35 ft. per 100 ft. We observe that this intersects the base 40 ft. within the outer toe of the bank slope. If the plane of saturation was 33 ft. per 100, it would just reach the outer toe. It would be advisable to enlarge this section by adding a 10-ft. berm at the 50-ft.level, having a slope not less than 3 on 1 for the up-stream face, and two 15-ft. berms on the down-stream face, having slopes $2\frac{1}{2}$ on 1. The additional width of base due to these modifications in our profile amounts to 65 ft., giving a total base width of 587 ft., and increasing the factor of safety from 10 to 13. It should be remembered that if the bank becomes saturated this factor of safety may be reduced 50%, the coefficient of moist clay being 0.50.

The loss of head due to a core wall of masonry, as designed for the New Croton Dam, was assumed by the board of experts to be 21 ft., or 17% of the depth of water in full reservoir. It has been stated by several authorities that the primary object of a masonry core wall is to afford a water-tight cut-off to any water of percolation which may reach it through the upper half of the embankment. It appears that absolute water-tightness in the core wall is not obtained, although the core walls of the Croton dams are said to be "the very best quality of rubble masonry that can be made."

Mr. W. W. Follett, who is reported to have had considerable experience in building earth dams, and who has made some valuable suggestions thereupon, is emphatic in saying,

> that the junction of earth and masonry forms a weak point, that either a puddle or masonry core in an earthen dam is an element of weakness rather than strength.

He also thinks the usual manner of segregating and depositing materials different in density and weight, and thus subject to differ-

ent amounts of settlement, as bad a form of construction as could be devised.

Core walls may prevent "free passage of water" and "excessive seepage," but are nevertheless of doubtful expediency.

Earthwork Slips and Drainage.

Mr. John Newman, in his admirable treatise on "Earthwork Slips and Subsidences upon Public Works," classifies and enumerates slips as follows:

Natural causes, 7.
Artificial causes, 31.
Additional causes due to impounded water, 7.

After describing each cause he presents 39 different means used to prevent such slips and describes methods of making repairs.

Mr. Wm. L. Strange has had such a large and valuable experience and has set forth so carefully and lucidly both the principles and practice of earth dam construction, that the writer takes pleasure in again quoting him on the subject of *drainage*, of which he is an ardent advocate. He says that,

> thorough drainage of the base of a dam is a matter of vital necessity, for notwithstanding all precautions, some water will certainly pass through the puddle.

It is at the junction of the dam with the ground that the maximum amount of leakage may be expected. The percolating water should be gotten out as quickly as possible. The whole method of dealing with slips may be summed up in one word—*drainage*.

The proper presentation of these two phases of our subject would in itself require a volume. The interested reader is therefore referred to the different authorities and writers cited in Appendix II.

Jerome Park Reservoir Embankments.

The Jerome Park reservoir is an artificial basin involving the excavation and removal of large quantities of soil, and the erection of long embankments with masonry core walls, partly founded on rock and partly on sand. The plan and specifications call for an embankment 20 ft. wide on top, with both slopes 2 on 1, and provide for lining the inner slope with brick or stone laid in concrete, and for covering the bottom with concrete laid on good earth compacted by rolling.

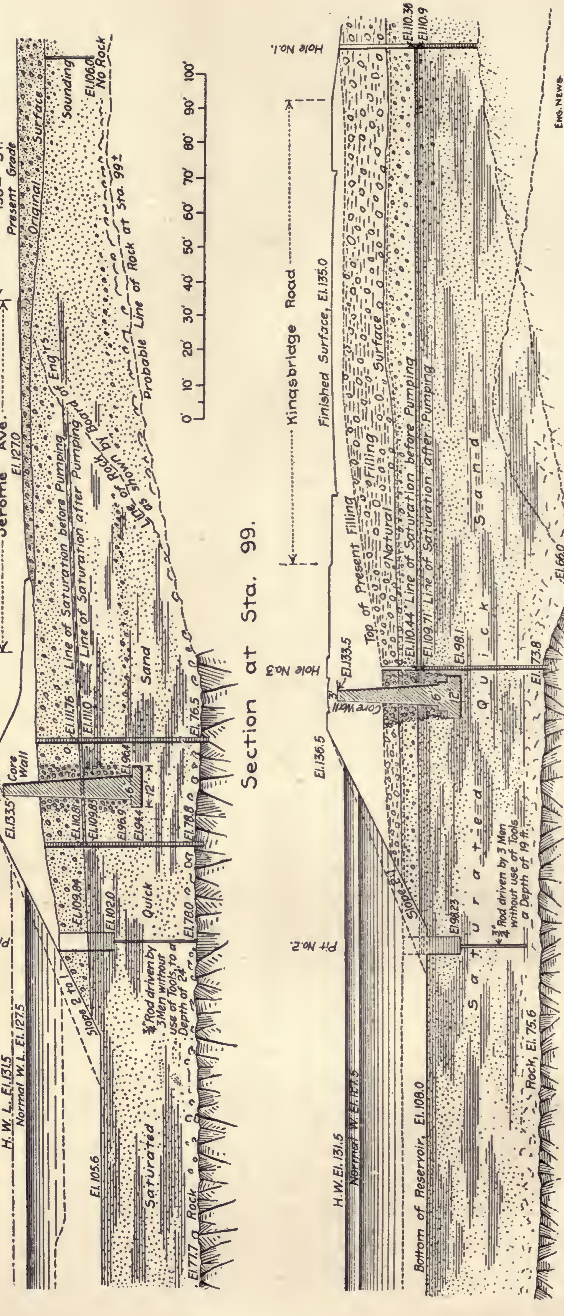


FIG. 20.—GRAPHICAL EXHIBIT OF STUDIES OF JEROME PARK RESERVOIR EMBANKMENT.

Wherever bed rock was not considered too deep below the surface the core walls were built upon it. In other places the foundation was placed 8 to 10 ft. below the bottom of the reservoir and rested upon the sand.

It appears that the plans of the Jerome Park embankment were changed from their original design, prior to the report of the board of experts, on account of two alleged defects, namely, "cracks in the core wall" and "foundation of quicksand," and incidentally on account of the supposed instability of the inner bank.

In describing the materials on which these embankments rest the experts remarked

that all these fine sands are unstable when mechanically agitated in an excess of water, and that they all settle in a firm and compact mass under the water when the agitation ceases. That they are quite unlike the true quicksands whose particles are of impalpable fineness and which are "quick" or unstable under water.

Fig. 20 is a graphic exhibit of the results of tests made at "Station 76 + 20," and at "Station 99," to determine the flow line of water in the sand strata underlying the embankment and bottom of the Jerome Park reservoir.

The experts reported that there was no possible danger of sliding or sloughing of the bank; that the utmost that could be expected would be the percolation of a small amount of water through the embankment and the earth; and that this would be carried off by the sewers in the adjacent avenues; that a large expenditure to prevent such seepage would not be warranted nor advisable.

In concluding their report, however, they recommended changing the inner slope of 2 on 1 to 2½ on 1, and doubling the thickness of the concrete lining at the foot of the slope to preclude all possibility of the sliding or the slipping of the inner bank in case of the water being lowered rapidly in the reservoir.

Mr. W. R. Hill, then chief engineer of the Croton Aqueduct Commission, favored extending the core walls to solid rock. He took exception to the manner of obtaining samples of sand by means of pipe and force-jet of water, claiming that only the coarsest sand was obtained for examination. He did not consider fine sand through which three men could run a ¾-in. rod 19 and 20 ft. to rock without use of a hammer, very stable material upon which to build a wall.

North Dike of the Wachusett Reservoir, Boston.

The North Dike of the Wachusett Reservoir is another large public work in progress at the present time. It is of somewhat unusual design and the preliminary investigations and experiments which led to its adoption are interesting in the extreme.*

The area to be explored in determining the best location for the dike was great, and the preliminary investigations conducted by means of wash drill borings, very extensive. A total of 1,131 borings were made to an average depth of 83 ft., the maximum depth being 286 ft. The materials were classified largely by the appearance of the samples, though chemical and filtration tests were also made. The plane of the ground water was from 35 to 50 ft. below the surface, and the action of the water-jet indicated in a measure the degree of permeability of the strata.

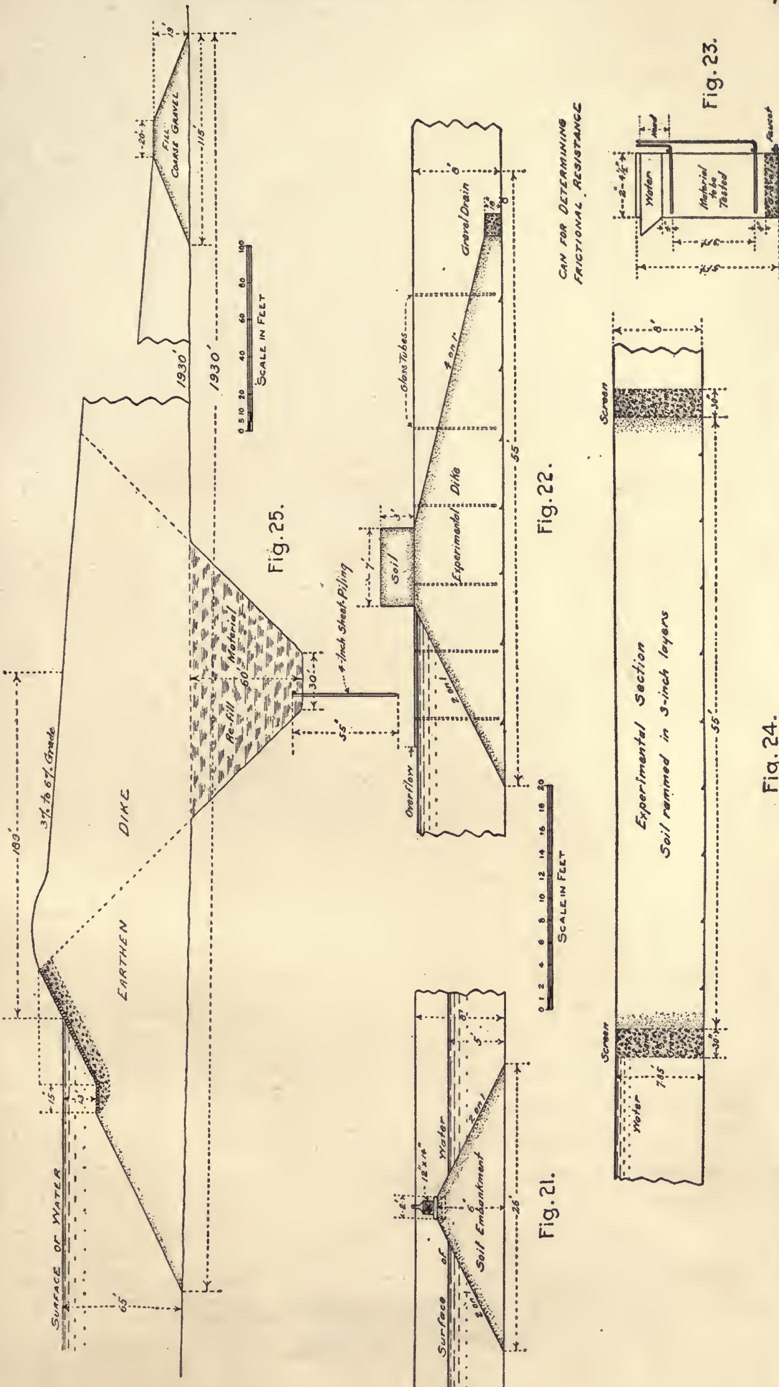
In addition to these tests experimental dikes of different materials, and deposited in different ways, were made in a wooden tank 6 ft. wide, 8 ft. high and 60 ft. long. The stability of soils when in contact with water was experimented with, as shown in Fig. 21, in the following manner:

An embankment (Fig. 21) was constructed in the tank of the material to be experimented with, 2 ft. wide on top, 6 ft. high, with slopes 2 on 1, and water admitted on both sides to a depth of 5 ft. The top was covered with 4-in. planks 2 ft. long and pressure applied by means of two jack screws resting upon a cross beam on top of the planks.

With a pressure of three tons per square foot, the 4-in.planks were forced down into the embankment a little more than 6 ins., resulting in a very slight bulging of the slopes a little below the water level. Immediately under the planks the soil became hard and compact. A man's weight pushed a sharp steel rod, $\frac{3}{4}$-in. in diameter, only 6 to 8 ins. into the embankment where the pressure was applied, while outside of this area the rod was easily pushed to the bottom of the tank.

These results corroborate in a general way the practical experience of the author, both in compressed embankments, where he found it necessary to use a pick vigorously to loosen the material of which they were composed, and in embankments made by mere-

*This work is very fully described in the Annual Reports of the Metropolitan Water Board of Boston: and by Mr. F. P. Stearns, Chief Engineer of the Metropolitan Water and Sewerage Board, in the Proceedings of the American Society of Civil Engineers for April, 1902. The latter description was reprinted, with the omission of some of the illustrations, in Engineering News for May, 8, 1902.



FIGS. 21 TO 24.—EXPERIMENTAL DIKES AND CYLINDER EMPLOYED IN STUDIES FOR THE NORTH DIKE OF THE WACHUSETT RESERVOIR; AND (FIG. 25) CROSS-SECTION OF THE DIKE.

ly dumping the material from a track, in which case the earth is so slightly compressed that an excavation is easily made with a shovel.

The difference in the coefficient of friction of the same material when dry and when wet greatly modifies the form of slope. The harder and looser the particles, the *straighter* will be the slope line in excavation and slips. The greater the cohesion of the earth, the *more curved* will be the slope, assuming a parabolic curve near the top—the true form of equilibrium.

RATE OF FILTRATION.—The rate of filtration through different soils was experimented with by forming a dike in the tank previously mentioned, as shown in Fig. 22.

The dike was made full 8 ft. high, 7 ft. wide on top, with a slope on the up-stream side of 2 on 1, and on the down-stream side 4 on 1. This gave a base width of 55 ft. Immediately over the top of the dike there was placed 3 ft. of soil to slightly consolidate the top of the bank and permit the filling of the tank to the top without overflowing the dike. The water pressure in different parts of the dike was determined by placing horizontal pipes through the soil cross-wise of the tank. These pipes were perforated and covered with wire gauze, being connected to vertical glass tubes at their ends. The end of the slope on the down-stream side terminated in a box having perforated sides and filled with gravel, thus enabling the water to percolate and filter out of the bank without carrying the soil with it.

When the soil was shoveled loosely into the tank, without consolidation of any kind, it settled on becoming saturated and became quite compact. It took five days for the water to appear in the sixth gage pipe near the lower end of the tank. After the pressure, which was maintained constant, had been on for several weeks, the seepage amounted to one gallon in 22 minutes. When the soil was deposited by shoveling into the water, the seepage amounted to one gallon in 34 minutes.

The relative filtering capacities of soils and sands were thought to be better determined by the use of galvanized iron cylinders of known areas.

Fig. 23 shows one of the cylinders. These latter experiments confirmed those previously made at Lawrence, by Mr. Allen Hazen, for the Massachusetts State Board of Health. They showed that the loss of head was directly proportioned to the quantity of water

filtered and that the quantity filtered will vary as the square of the diameter of the *effective size* of the grains of the filtering material.*

The material classed as "permeable" at the North Dike of the Wachusett Reservoir has an effective diameter of about 0.20 mm. A few results are given in the following table:

Amount of Filtration in Gallons per Day, Through an Area of 10,000 Sq. Ft., With a Loss of Head or Slope of 1 ft. in 10 ft.

	Material.	Unit ratios.	U. S. gallons.
(1)	Soil	1	510
(2)	Very fine sand	14	7,200
(3)	Fine sand	176	90,000
(4)	Medium sand	784	400,000
(5)	Coarse sand	4,353	2,200,000

To be sure that the accumulation of air in the small interstices of the *soil* was not the cause of the greatly reduced filtration through it, another series of experiments was conducted in the wooden tank, as shown in Fig. 24.

A pair of screens was placed near each end of the tank, filled with porous material, sand and gravel, and the 50-ft. space between filled with soil. The soil was rammed in 3-in. layers, and special care taken to prevent water from following along the sides and bottom of the tank. One end was filled with water to near the top, while the other end gave a free outlet.

After this experiment had been continued for more than a month, the amount of seepage averaged 1.7 gallons per 24 hours, or about 32 drops per minute.

Filtration tests were also made through soil under 150 ft. head, or 65 lbs. per sq. in., with results not materially different, it is stated, from those already given. The soil used in all these tests contained from 4 to 8% by weight of organic matter. This was burned and similar tests made with the incinerated soil, resulting in an increase of about 20% more seepage water.

PERMANENCE OF SOILS.—This last material experimented with suggests the subject of *permanence* of soils. This was reported upon separately and independently by Mr. Allen Hazen and Prof. W. O. Crosby. These experts agreed in their conclusion, stating

that the process of oxidation below the line of saturation would be extremely slow, requiring many thousands of years for the complete removal

*By effective size of sand grains is meant such size of grain that 10% by weight of the particles are smaller, and 90% larger than itself; or, to express it a little differently, the effective size is equal to a sphere the volume of which is greater than $^1/_{10}$ that forming the weight and is less than $^9/_{10}$ that forming the weight.

> of all the organic matter, and that the tightness of the bank would not be materially affected by any changes which are likely to occur.

It has been remarked,

> that of all the materials used in the construction of dams, *earth* is physically the least destructible of any. The other materials are all subject to more or less disintegration, or change in one form or another, and in earth they reach their ultimate and most lasting form.

In speaking of the North Dike of the Wachusett Reservoir, Mr. Stearns remarked that,

> it was evident by the application of Mr. Hazen's formula for the flow of water through sands and gravels, that the very fine sands found at a considerable depth below the surface would not permit enough water to pass through them if a dike of great width were constructed, to cause a serious loss of water, and it was also found that the soil, which contained not only the fine particles or organic matter, but also a very considerable amount of fine comminuted particles, which the geologist has termed "rock flour," would be sufficiently impermeable to be used as a substitute for clay puddle.

Fig. 25 shows the maximum section of the North Dike with its cut-off trench. The quantities and estimated cost of the completed structure are given in the table herewith:

			Cost,	
Work.	Quantities.	Unit price.	Actual.	Per cent. total.
Soil	5,250,000 cu. yds.	$0.05	$262,500	34.7
Cut-off trench	542,000 "	.20	108,400	19.3
Borrowed earth and gravel	200,000 "	.20	40,000	
Slope paving	50,000 "	2.20	110,000	14.6
Sheet-piling, pumping, etc.			117,000	15.5
Engineering and preliminary investigations 16%			120,000	15.9
Total cost			$757,900	100.0

Druid Lake Dam, Baltimore, Md.

Another very interesting and instructive example of high earth dam construction is that of the Druid Lake Reservoir embankment, Baltimore, Md.

This dam was built under the supervision of Mr. Robt. K. Martin. Construction was begun in 1864, and the dam was finished in 1870. Mr. Alfred M. Quick, present chief engineer of the water-works of the City of Baltimore has given a very lucid description of this work in Engineering News of Feb. 20, 1902.

Fig. 26 is a cross-section of this dam, showing the method of construction so clearly as to scarcely need further description. The banks D-D on either side of the central puddle wall were car-

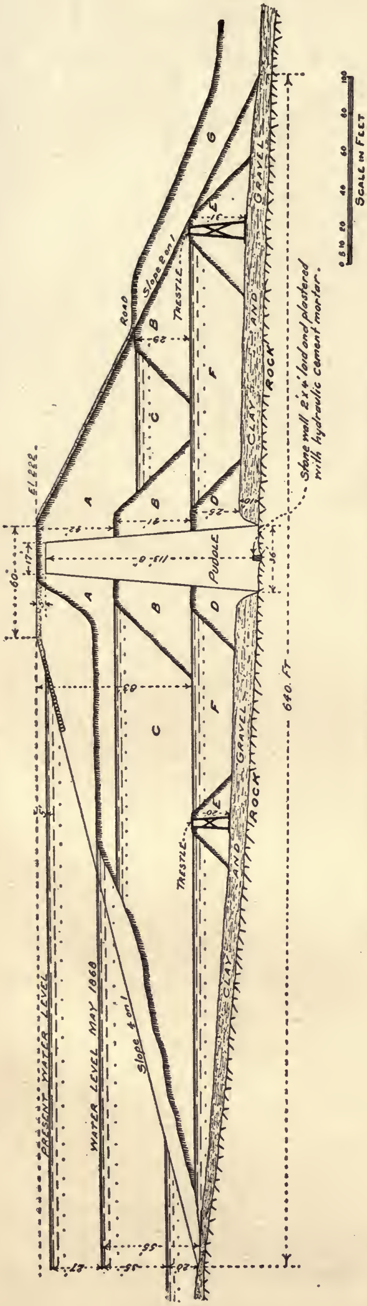


FIG. 26.—WORKING CROSS-SECTION OF DRUID LAKE DAM.

ried up in 6-in. layers with horses and carts, and kept about 2 ft. higher than the puddle trench, which always contained water. The banks E-E were made of dumped material, after which the basins F-F were first filled with water and finally filled by dumping material into the water from tracks being moved in toward the center.

After reaching the top of this fill, banks B-B-B were built up in layers similar to D-D. The second set of basins C-C were then filled in a manner similar to F-F. The remaining portion A-A was constructed in layers like D-D and B-B, with the addition of compacting each layer with a heavy roller.

Finally the inner face slope was carred up in 3-in. layers and thoroughly rolled, after which 2 ft. of "good puddle" was put upon the inner slope the latter was rip-rapped, the crown covered with gravel and the rear slope sodded.

Some years after completion, a driveway was built along the outer slope, as shown, which had a tendency to strengthen the dam, though not designed expressly for that purpose.

It is of interest to know that the influent, effluent and drain pipes were originally constructed through or under the embankment. These pipes were laid upon solid earth, and where they passed

through the puddle wall were supported upon stone piers 6 ft apart. As might be expected, they soon cracked badly and were finally abandoned, new ones being placed in the original ground at the south side of the lake. Mr. Quick states that so far as is known there has never been any evidence of a leak through the embankment during these 32 years of service.

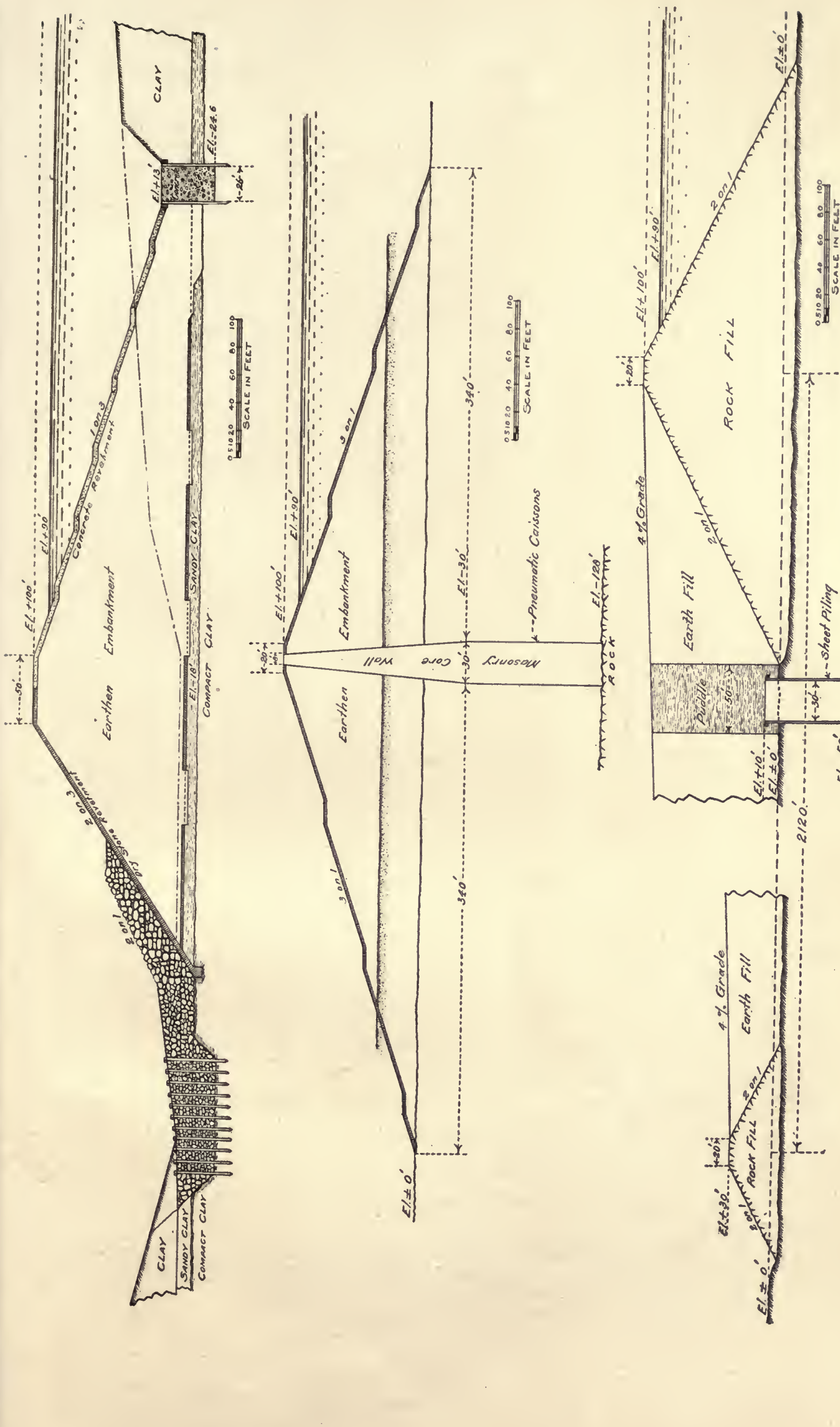
New Types of Dams; Bohio, Panama Canal.

A brief description will now be given of three different dams designed for Bohio, on the proposed Panama Canal. Mr. George S. Morison's paper before the American Society of Civil Engineers, on "The Bohio Dam," and the discussion thereon, especially that by Mr. F. P. Stearns, were quite fully reported in Engineering News for March 13 and May 8, 1902. In constructing the Panama Canal it will be necessary to impound the waters of the Chagres River, near Bohio, to maintain the summit level of this canal and supply water for lockage.

THE FRENCH DESIGN.—Fig. 27 is an enlarged section of the original design of the new French Co. This design has no core wall, but at the up-stream toe a concrete wall was to be built across the river between the two lines of sheet-piling. At the down-stream toe a large amount of riprap was to be placed to prevent destruction of the dam during construction. In this case it would be necessary to construct a temporary dam above and also to use the excavation for the locks as a flood spillway. This method would involve considerable risk to the work, on account of the large volume of flood waters it might be necessary to take care of during construction.

ISTHMIAN CANAL COMMISSION.—The dam proposed by the Isthmian Canal Commission is shown by Fig. 28. This was designed to be an absolutely water-tight closure of the geological valley, by using a masonry core wall carried down to bed rock. The maximum depth being 129 ft., it was planned to rest the concrete wall on a series of pneumatic caissons reaching to rock. The spaces between the caissons would be closed and made water-tight. Both slopes of the earth embankment were to have horizontal benches and be revetted with loose rock.

MR. MORISON'S DESIGN.—To appreciate fully the object and aim of the third design, Fig. 29, which may be called a new type, although similar in many respects to the North Dike of the Wachusett reservoir already illustrated and described, it should be stated that the equalized flow of the Chagres River is put at 1,000 cu. ft.



FIGS. 27 TO 29.—DESIGNS FOR THE BOHIO DAM, PANAMA CANAL.

per sec. Of this quantity it is estimated that 500 cu. ft. would be needed for lockage and 200 cu. ft. for evaporation. This leaves 300 cu. ft. per sec. available for seepage and other losses or to be wasted.

It will thus be seen that a scarcity of water is not in this instance a condition demanding an absolutely water-tight dam. The amount of seepage permissible without endangering the stability of the structure is the real point now to be discussed.

The third design, which was proposed by Mr. Morison, is shown by Fig. 29. The topography and configuration of this dam site is not unlike that of the San Leandro Dam, California, soon to be described, while the yeneral design is similar, as has been remarked, to the North Dike of the Wachusett Reservoir.

This third design contemplates a compound structure, formed by two rock-fill dams situated about 2,120 ft. apart, with the intervening space filled with loose rock, earth and other availible material. Immediately below the upper and higher rock-fill dam, it is proposed to place across the canyon a puddle wall 50 ft. in width, resting over two lines of sheet-piling 30 ft. apart. This piling would probably not reach farther than 50 ft. below tidewater, the solid rock floor being about 100 ft. deeper.

Mr. Morison made use of Mr. Hazen's filtration formula for estimating the rate and quantity of seepage through the permeable strata below the dam. This formula is:

$$V = cd^2 \frac{h}{l} \cdot \frac{t + 10^\circ}{60} \qquad \text{where}$$

V=rate of flow in meters per day through the whole section.

c=constant varying from 450 to 1,200, according to cleanness of the sand.

d="effective size" of sand in mm.

h=head in feet.

l=length or distance water must pass.

t=temperature of the water (Fahr.)

This formula should be used only when the *effective sizes* of sands are from 0.10 to 3.0 mm. and with *uniformity coefficients* below 5.0*

Mr. Morison used the following values: C=1,000; d=1.0 mm.; h=90 ft.; l=2,500 ft.; t=90°; for the solution of this problem, and

*The term "uniformity coefficient" is used to designate the ratio of the size of the grain which has 60% of the sample finer than itself to the size which has 10% finer than itself. The method of determining the size of sand grains and their uniformity coefficients, is fully explained in Appendix 3 of Mr. Hazen's book on "The Filtration of Public Water Supplies."

obtained a velocity of 0.002 ft. per sec. The bed of sand and gravel was assumed to have a sectional area of 20,000 sq. ft. for 2,500 ft. in length. This gives a seepage of 40 cu. ft. per sec.

It is believed that the above rate of 0.002 ft. per sec., equivalent to 1 3-8 ins. per minute, or 7 ft. per hour, is not sufficient to move any of the material. The velocity of water percolating through sand is found to vary directly as the head and inversely as the distance.

The value of "c" in the formula is larger for sands of filters favorable for flow, and smaller for compacted materials and dams.

Mr. Morison thought it might be nearer the actual conditions to assume d=0.50 mm.; c=500; and l=5,000 ft.; in which case the seepage would only amount to 2.5 ft. per sec. In this last assumption the "effective size" of sand grains is 2½ times that classed as "permeable material" at the North Dike of the Wachusett Reservoir.

Prof. Philipp Forchheimer, of Gratz, Austria, recommends the use of the formula,

$$\frac{h}{l} = aV + bV^2$$

for the percolation through soils between loam and loamy sand. Sellheim, Masoni, Smreker, Kröber and other authorities on filtration use still other formulas, to which the reader and student is referred for further research.

The writer, having had occasion in his professional practice to study quite carefully the subject of ground waters, and their percolation or flow through different classes of materials and under varying conditions, is of the opinion that rarely does the cross-section of a stream-channel, filled with sand, gravel and debris, present, even approximately, a homogeneous or uniform mass; and that there are, almost without exception, strata of material much coarser and more porous than the general average. In other words, that it is extremely difficult to arrive at a uniformity coefficient. It is unwise to place much reliance upon an estimated flow where this is the case. The formula may be used with confidence where the layers are artificially made, and where there is no uncertainty regarding the uniform character of the material. In most natural channels there are distinct lines of flow, and under considerable hydrostatic head or pressure these lines of flow would surely enlarge. There is a wide difference between permissible and dangerously excessive percolation through an earth embankment. The local features, economical considerations and magni-

tude of the risks, all bear upon this question and must be considered for each particular case.

It is of interest to compare the estimated cost of the three designs proposed for the Bohio Dam, based upon the same unit prices, as follows:

French Engineers' design	$3,500,000
Isthmian Canal Commissioners' design	8,000,000
Mr. Morison's design.	2,500,000

No comments will be made upon these figures, further than to remark that the successful building of a stable dam, accomplished by the use of an excessive quantity of materials and at a cost beyond reasonable requirements, is mainly instructive as illustrating "how not to do it." It is creditable to execute substantial works at a reasonable cost, but it reflects no credit upon any one to construct them regardless of expense.

Combined Rock-fill and Earth Dam.

Fig. 30 shows a section of the Upper Pecos River Dam near Eddy, N. M.

This dam is quite fully described by Mr. Jas. D. Schuyler, in his recent book on "Reservoirs for Irrigation, Water-Power and Domestic Water-Supply," and need not be mentioned in this paper, further than to call attention to the combination of rock-fill and earth which constitutes its particular type of construction. This type of dam is believed to be for manv localities a very good one, but up to the present time has only been adopted for dams of moderate height, under 60 ft.

The San Leandro Dam, California.

A section of the San Leandro Dam, near Oakland, Cal., is shown by Fig. 31. This section was supplied by Mr. W. F. Boardman, hydraulic engineer, who superintended the construction of the dam, from his own private notes and data. It differs materially from sections heretofore published, and is 5 ft. higher, thus making it rank as the highest earth dam in the world of which we have an authentic record.

The dam was commenced in 1874, and brought up to a height of 115 ft. above the bed of the creek in 1898. At the present time it is 500 ft. in length on the crest and 28 ft. wide. The original width of the ravine at the base of the dam was 66 ft. The present width of base from toe to toe of slopes is 1,700 ft. The height of

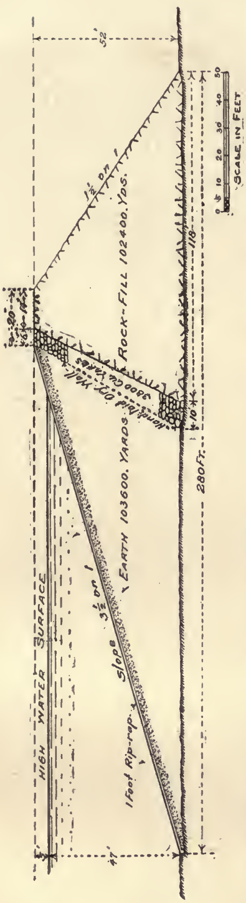


FIG. 30.—CROSS-SECTION OF UPPER PECOS RIVER DAM; COMBINED ROCK FILL AND EARTH.

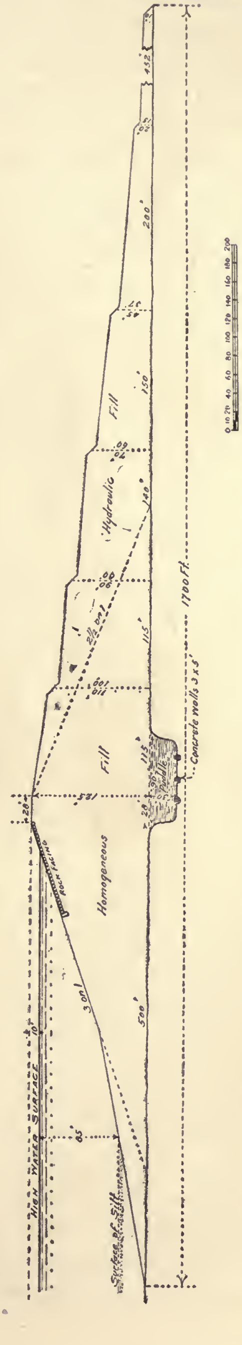


FIG. 31.—DEVELOPED SECTION OF SAN LEANDRO DAM.

embankment above the original surface is 125 ft., with a puddle trench extending 30 ft. below.

All that portion of the dam within a slope of $2\frac{1}{2}$ on 1 at the rear and 3 on 1 at the face is built of choice material, carefully selected and put in with great care. The portion outside of the $2\frac{1}{2}$ on 1 slope line at the down-stream side of the dam, was *sluiced in* from the adjacent hills regardless of its character, and is composed of ordinary soil containing more or less rock.

This process of sluicing was carried on during the rainy season, when there was an abundance of water, and it was intended to be continued until the canyon below the dam had been filled to an average slope of 6.7 on 1 at the rear of the dam. It was thought that the location was particularly favorable for this kind of construction, the original intention being to raise the dam from time to time, not only to increase the storage as the demand for water increased, but to meet the annual loss in capacity caused by the silting up of the reservoir basin. The latter has amounted to about 1 ft. in depth per annum.

METHOD OF CONSTRUCTION.—Under the main body of the dam, the surface was stripped of all sediment, sand, gravel and vegetable matter. Choice material, carefully selected, was then brought by carts and wagons and evenly distributed over the surface in layers about 1 ft. or less in thickness. This was sprinkled with just enough water to make it pack well, not enough to make it like mud. During construction a band of horses was led by a boy on horseback over the entire work, to compact the materials and assist in making the dam one homogeneous mass. No rollers were used on this dam.

The central trench was cut 30 ft. below the original bed of the creek. In the bottom of this trench three secondary trenches, 3 ft. wide by 3 ft. deep, were made and filled with concrete. These concrete walls were carried up 2 ft. above the general floor of the trench, to break the continuity of its surface.

The original wasteway, constructed at the north end of the dam, has been practically abandoned, having been substituted by a tunnel of larger capacity. The original wasteway was excavated in the bed rock of the natural hillside, and although lined with masonry, is not in the best condition. The author considers its location an objectionable feature, as menacing the safety of the dam, and thinks it should be permanently closed.

A wasteway tunnel, 1,487 ft. in length, was constructed in 1888,

through a ridge extending north of the dam. This has a sectional area of about 10x10 ft., lined with brick masonry throughout, having a grade of $2\frac{1}{2}\%$.

The criticism might be made of the tunnel that it is faulty in design at the entry or reservoir end, where the water must first fall over a high spillway wall, aerating the water before entering the tunnel proper. The water even then has not easy access to the tunnel, and no adequate arrangements have been made for ventilation, so as to insure the utilization of its maximum capacity. The maximum depth of water in the reservoir is about 85 ft., and the full capacity 689,000,000 cu. ft. of water. The catchment area is 43 square miles, and the surface of the reservoir when full 436 acres. The outlet pipes are placed in two tunnels at different elevations through the ridge north of the dam. There are no culverts or pipes extending through the body of the dam itself.

Hydraulic-fill Dams.

No discussion of earth dams would be complete without some reference being made to the novel type of construction developed in western America in recent years, by which railroad embankments and water-tight dams are built up by the sole agency of water. The water for this purpose is usually delivered under high pressure, as it is generally convenient to make it first perform the work of loosening the earth and rock in the borrow pit, as well as subsequently to transport them to the embankment, and there to sort and deposit them and finally part company with them after compacting them solidly in place, even more firmly than if compressed by heavy rollers. Sometimes, however, water is delivered to the borrow pit without pressure, in which event the materials must be loosened by the plow or by pick and shovel by the process called ground sluicing in placer mining parlance.

An abundance of water delivered by gravity under high pressure is usually regarded as one of the essential factors in hydraulic-fill dam building, but it is not essential that there be a large continuous flow. The Lake Frances Dam, recently constructed for the Bay Counties Co., of California, by J. D. Schuyler, is 75 ft. high, 1,340 ft. long on top, and contains 280,000 cu. yds. The dam was built up by materials sluiced by water that was forced by a centrifugal pump through a 12-in. pipe and 3-in. nozzle, against a high bank, whence the materials were torn and conveyed by the water through flumes and pipes to the dam. About 6 cu. ft. per sec. of

water was thus used, and at one stage of the work the supply stream was reduced to less than 0.1 ft. per sec., the water being gathered in a pond and pumped over and over again.

The chapter on hydraulic-fill dams in Mr. Schuyler's book on "Reservoirs for Irrigation" will be found to contain matter on the subject interesting to those who desire to pursue it further, and the reader is again referred to that work.

An Impervious Diaphragm in Earth Dams.

As a result of the recent extended discussion concerning the design of the New Croton Dam and the Jerome Park Reservoir embankments, the Engineering News of Feb. 20, 1902, contained a very suggestive editorial entitled, "Concerning the Design of Earth Dams and Reservoir Embankments." The opinion is given that no type of structure that man builds to confine water can compare in permanence with earth dams, after which the following pertinent questions are asked:

1. How shall an earth dam be made water-tight?
2. What is the office and purpose of the masonry core wall?
3. Would not a water-proof diaphragm of some kind be better than a core wall of either masonry or puddle?

The article then suggests a number of designs of diaphragm construction, with a special view of obtaining absolute water-tightness, by use of asphaltum, cement-mortar, steel plates, etc. Special emphasis was put upon the *principle* of constructing a waterproof diaphragm. The matter of relative cost is advanced as an argument in favor of the diaphragm principle as against the usual orthodox method. The saving in cost is to be accomplished by the use of inferior materials and less care in the handling of them, or by both. It is suggested that almost any kind of material available, rock, sand or gravel, will answer every purpose where good earth is not to be found. Further, that this material may be dumped from the carts, cars or cableways, or be placed by the hydraulic-fill method.

The writer believes the diaphragm method of construction may have some merits, but that it is attended by the very great risk of neglecting principles most vitally important to the successful construction of high earth dams, which will now be formulated and advanced, as follows:

CHAPTER VI.

Conclusions.

The writer in concluding this study wishes to emphasize certain principles and apparently minor details of construction, which from observation and personal experience, seem to him of vital importance.

He believes firmly in the truth contained in the following remarks by Mr. Desmond FitzGerald, of Boston, germane to this subject:

An engineer must be guided by local conditions and the resources at his command in building reservoir embankments. His design must be largely affected by the nature of the materials. There are certain *general principles*, however, which must be observed and which will be applied by an engineer of skill, judgment and experience to whatever design he may adopt. It is in the application of these principles that the services of the professional man becomes valuable, and it is from a lack of them, that there have been so many failures.

The details and principles of construction, relating to high earth dams, may be summarized or stated in order of their application, as follows:

(1) Select a firm, dry, impermeable foundation, or make it so by excavation and drainage. All alluvial soil containing organic matter and all porous materials should be excavated and removed from the dam site when practicable; that is, where the depth to a suitable impermeable foundation is not prohibitive by reason of excessive cost.

Wherever springs of water appear, they must be carried outside the lines of the embankment by means of bed rock drains, or a system of pipes so laid and embedded as to be permanent and effective.

The drainage system must be so designed as to prevent the infiltration of water upward and into the lower half of the embankment, and at the same time insure free and speedy outlet for any seepage water passing the upper half. All drains should be placed upon bed rock or in the natural formation selected for the foundation of the superstructure. They should be constructed in such a manner as to prevent the flow of water outside the channel provided for it, and also prevent any enlargement of the channel itself.

To this end, cement, mortar, broken stone, and good gravel puddle are the materials best suited for this purpose.

(2) Unite the body of the embankment to the natural foundation by means of an impervious material, durable and yet sufficiently elastic to bond the two together. When the depth to a suitable foundation is great, a central trench excavated with sloping sides, extending to bed rock or other impervious formation, refilled with good puddling material, properly compacted, will suffice.

When clayey earth is scarce and expensive to obtain, a small amount of clay puddle confined between walls of brick, stone or concrete masonry, and extending well into the body of the embankment and so built as to avoid settlement, will prevent excessive seepage. This form of construction is not to be carried much above the original surface of the ground.

(3) The continuity of surfaces should always be broken, at the same time avoiding the formation of cavities and lines of cleavage. No excavation to be refilled should have vertical sides, and long continuous horizontal planes should be intercepted by wedge-shaped offsets, enabling the dovetailing of materials together.

All loose and seamy rock or other porous material should be removed, and where the refill is not the best for the purpose, mix the good and bad ingredients thoroughly, after which deposit in very thin layers.

(4) Make the dimensions and profile of dam with a factor of safety against sliding of not less than ten. The preliminary calculations for designing such a profile have been given on p. 42.

(5) Aim at as nearly a homogeneous mass in the body of the embankment as possible, thus avoiding unequal settlement and deformation. This manner of manipulating materials will eliminate many uncertain or unknown factors, but it means rigid inspection of the work and intelligent segregation of materials, no matter what method of transporting them may be adopted. The smaller the unit loads may be, the more easily a homogeneous distribution of materials will be obtained.

(6) Select earthy materials in preference to organic soils, with a view of such combination or proportion of different materials as will readily consolidate. *Consolidation is the most important process connected with the building of an earth dam.* The judicious use of soil containing a small percentage of organic matter may be permitted, however, when there is a lack of clayey material for mixing with sandy and porous earth materials. Such a mixture, properly

distributed and wetted, will consolidate well under heavy pressure and prove quite satisfactory.

(7) Consolidation being the most important process and the only safeguard against permeability and instability of form, use only the amount of water necessary to attain this. Too much or too little are equally bad and to be avoided. It is believed that only by experiment and experience is it possible to determine just the proper quantity of water to use with the different classes of materials and their varying conditions. In rolling and consolidating the bank, all portions that have a tendency to quake must be removed at once and replaced with material that will consolidate; it *must not* be covered up, no matter how small the area.

(8) In an artificial embankment for impounding water it is impracticable to place reliance upon time for consolidation; it *must* be effected by mechanical means. Again we repeat, that consolidation is the most vitally important operation connected with the building of an earth dam. When this is satisfactorily attained it is proof that the materials are suitable and that the other necessary details have been in a large measure complied with. Light rollers are worse than useless, being a positive harm, resulting in a smoothing or "ironing process," deceptive in appearance and detrimental in many ways.

The matter of supreme importance in the construction of earth dams is that the greatest consolidation possible be specified and effected. To this end it is necessary that heavy rollers be employed, and that such materials be selected as respond best to the treatment. There are certain kinds of earth materials which no amount of wetting and rolling will compact. These must be rejected as unfit for use in any portion of an earth dam. Let the design of the structure be ever so true to correct engineering principles, it is still necesssary to give untiring attention to the work of consolidation. It is therefore according to the design of a thoroughly compacted homogeneous mass, rather than to the suggested *diaphragm type,* to which modern practice should conform. This is in harmony with Nature's own methods, and in conformity to correct principles.

(9) Avoid placing pipes or culverts through any portion of the embankment. The writer considers it bad practice ever to place the outlet pipes through a high earth dam, and fails to see any necessity for so doing.

(10) The surface of the dam, both front and rear, must be suit-

ably protected against the deteriorating effects of the elements. This may include pitching the up-stream face, the riprap work at the toe of the inner slope, the roadway and covering of the crown, the sodding or other protection of the rear slope, and the construction of surface drains for the berms.

(11) Ample provision for automatic wasteways should be made for every dam, so that the embankment can never under any circumstances be over-topped by the impounded water. Earthquakes and seismic disturbances will produce no disastrous effects upon an earth dam. Its elasticity will resist the shock of water lashing backwards and forwards in the reservoir.

(12) Finally, provide for intelligent and honest supervision during construction, and insist upon proper care and maintenance ever afterwards.

APPENDIX I.

High Earth Dams.

Name of Dam or Reservoir.	Location.	—Embankment— Max. height, ft.	Top width, ft.	—Slopes— Water.	Rear.	Available depths, ft.
San Leandro	California	125	28	......	......	..
Tabeaud	California	123	20	3 on 1	2½ on 1	70
Druid Hill	Maryland	119	60	4 on 1	2 on 1	82
Dodder	Ireland	115	22	3½ on 1	3 on 1	..
Titicus Dam	New York	110	30	2 on 1	2½ on 1	..
Mudduk Tank	India	108	..	3 on 1	2½ on 1	..
Cummum Tank	India	102	..	3 on 1	1 on 1	90
Dale Dike	England	102	12	2½ on 1	2½ on 1	..
Marengo	Algeria	101	..	......	......	..
Torside	England	100	..	......	......	84
Yarrow	England	100	24	3 on 1	2 on 1	..
Honey Lake	California	96	20	3 on 1	2 on 1	..
Pilarcitos	California	95	25	2¾ on 1	2½ on 1	..
San Andres	California	95	25	3½ on 1	3 on 1	..
Temescal	California	95	12	3 on 1	2 on 1	..
Waghad	India	95	6	3 on 1	2 on 1	81
Bradfield	England	95	12	2½ on 1	2½ on 1	..
Oued Meurad	Algeria	95	..	......	......	..
St. Andrews	Ireland	93	25	......	......	..
Edgelaw	Scotland	93	..	3 on 1	2½ on 1	..
Woodhead	England	90	..	......	......	72
Tordoff	Scotland	85	10	3 on 1	2½ on 1	..
Naggar	India	84	..	......	......	..
Vahar	India	84	24	3 on 1	2½ on 1	..
Rosebery	Scotland	84	..	......	......	..
Atlanta	Georgia	82	40	......	......	..
Roddlesworth	England	80	16	3 on 1	2½ on 1	68
Gladhouse	Scotland	79	12	3 on 1	2½ on 1	68½
Rake	England	78	..	3 on 1	2 on 1	..
Silsden	England	78	..	3 on 1	2 on 1	..
Glencourse	Scotland	77	..	3 on 1	......	58
Leeshaw	England	77	..	......	......	..
Wayoh	England	76	22	3 on 1	2½ on 1	..
Ekruk Tank	India	76	20	3 on 1	2 on 1	65
Nehr	India	74	8	......	......	..
Middle Branch	New York	73	..	......	......	..
Leeming	Ireland	73	10	3 on 1	2 on 1	50
South Fork	Penna	72	20	2 on 1	1½ on 1	..
Anasagur	India	70	20	4 on 1	.....	..
Pangran	India	68	8	......	......	42
Harlaw	Scotland	67	..	......	......	64
Lough Vartry	Ireland	66	28	3 on 1	2½ on 1	60
La Mesa	California	66	20	1½ on 1	1½ on 1	60
Amsterdam	New York	65	..	......	......	
Mukti	India	65	10	3 on 1	2 on 1	41
Snake River	California	64	12	2 on 1	1½ on 1	..
Stubken	Ireland	63	24	3 on 1	2 on 1	..
Den of Ogil	Scotland	60	..	......	......	50
Loganlea	Scotland	59	10	3 on 1	2½ on 1	55
Ashti	India	58	6	3 on 1	2 on 1	42
Cedar Grove	New Jersey	55	18	3 on 1	2 on 1	50

APPENDIX.—II.

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